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NUCLEAR ENERGY AGENCY
COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

NEA/CSNI/R(2002)7/VOL1
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OECD-NEA WORKSHOP ON THE EVALUATION OF DEFECTS, REPAIR CRITERIA & METHODS OF REPAIR FOR CONCRETE STRUCTURES ON NUCLEAR POWER PLANTS

Hosted by GRS at the DIN Institute in Berlin, Germany

10th-11th April, 2002

English - Or. English

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ORGANISATION FOR ECONOMIC CO-OPERATION AND DEVELOPMENT

Pursuant to Article 1 of the Convention signed in Paris on 14th December 1960, and which came into force on 30th September 1961, the Organisation for Economic Co-operation and Development (OECD) shall promote policies designed:

- to achieve the highest sustainable economic growth and employment and a rising standard of living in Member countries, while maintaining financial stability, and thus to contribute to the development of the world economy;
- to contribute to sound economic expansion in Member as well as non-member countries in the process of economic development; and
- to contribute to the expansion of world trade on a multilateral, non-discriminatory basis in accordance with international obligations.

The original Member countries of the OECD are Austria, Belgium, Canada, Denmark, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, the Netherlands, Norway, Portugal, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The following countries became Members subsequently through accession at the dates indicated hereafter: Japan (28th April 1964), Finland (28th January 1969), Australia (7th June 1971), New Zealand (29th May 1973), Mexico (18th May 1994), the Czech Republic (21st December 1995), Hungary (7th May 1996), Poland (22nd November 1996), Korea (12th December 1996) and the Slovak Republic (14th December 2000). The Commission of the European Communities takes part in the work of the OECD (Article 13 of the OECD Convention).

NUCLEAR ENERGY AGENCY

The OECD Nuclear Energy Agency (NEA) was established on 1st February 1958 under the name of the OEEC European Nuclear Energy Agency. It received its present designation on 20th April 1972, when Japan became its first non-European full Member. NEA membership today consists of 27 OECD Member countries: Australia, Austria, Belgium, Canada, Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Japan, Luxembourg, Mexico, the Netherlands, Norway, Portugal, Republic of Korea, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The Commission of the European Communities also takes part in the work of the Agency.

The mission of the NEA is:

- to assist its Member countries in maintaining and further developing, through international co-operation, the scientific, technological and legal bases required for a safe, environmentally friendly and economical use of nuclear energy for peaceful purposes, as well as
- to provide authoritative assessments and to forge common understandings on key issues, as input to government decisions on nuclear energy policy and to broader OECD policy analyses in areas such as energy and sustainable development.

Specific areas of competence of the NEA include safety and regulation of nuclear activities, radioactive waste management, radiological protection, nuclear science, economic and technical analyses of the nuclear fuel cycle, nuclear law and liability, and public information. The NEA Data Bank provides nuclear data and computer program services for participating countries.

In these and related tasks, the NEA works in close collaboration with the International Atomic Energy Agency in Vienna, with which it has a Co-operation Agreement, as well as with other international organisations in the nuclear field.

COMMITTEE ON NUCLEAR REGULATORY ACTIVITIES

The Committee on Nuclear Regulatory Activities (CNRA) of the OECD Nuclear Energy Agency (NEA) is an international committee made up primarily of senior nuclear regulators. It was set up in 1989 as a forum for the exchange of information and experience among regulatory organisations and for the review of developments which could affect regulatory requirements.

The Committee is responsible for the programme of the NEA, concerning the regulation, licensing and inspection of nuclear installations. The Committee reviews developments which could affect regulatory requirements with the objective of providing members with an understanding of the motivation for new regulatory requirements under consideration and an opportunity to offer suggestions that might improve them or avoid disparities among Member Countries. In particular, the Committee reviews current practices and operating experience.

The Committee focuses primarily on power reactors and other nuclear installations currently being built and operated. It also may consider the regulatory implications of new designs of power reactors and other types of nuclear installations.

In implementing its programme, CNRA establishes co-operative mechanisms with NEA's Committee on the Safety of Nuclear Installations (CSNI), responsible for co-ordinating the activities of the Agency concerning the technical aspects of design, construction and operation of nuclear installations insofar as they affect the safety of such installations. It also co-operates with NEA's Committee on Radiation Protection and Public Health (CRPPH) and NEA's Radioactive Waste Management Committee (RWMC) on matters of common interest.

COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

The NEA Committee on the Safety of Nuclear Installations (CSNI) is an international committee made up of scientists and engineers. It was set up in 1973 to develop and co-ordinate the activities of the Nuclear Energy Agency concerning the technical aspects of the design, construction and operation of nuclear installations insofar as they affect the safety of such installations. The Committee's purpose is to foster international co-operation in nuclear safety amongst the OECD Member countries.

CSNI constitutes a forum for the exchange of technical information and for collaboration between organisations which can contribute, from their respective backgrounds in research, development, engineering or regulation, to these activities and to the definition of its programme of work. It also reviews the state of knowledge on selected topics of nuclear safety technology and safety assessment, including operating experience. It initiates and conducts programmes identified by these reviews and assessments in order to overcome discrepancies, develop improvements and reach international consensus in different projects and International Standard Problems, and assists in the feedback of the results to participating organisations. Full use is also made of traditional methods of co-operation, such as information exchanges, establishment of working groups and organisation of conferences and specialist meeting.

The greater part of CSNI's current programme of work is concerned with safety technology of water reactors. The principal areas covered are operating experience and the human factor, reactor coolant system behaviour, various aspects of reactor component integrity, the phenomenology of radioactive releases in reactor accidents and their confinement, containment performance, risk assessment and severe accidents. The Committee also studies the safety of the fuel cycle, conducts periodic surveys of reactor safety research programmes and operates an international mechanism for exchanging reports on nuclear power plant incidents.

In implementing its programme, CSNI establishes co-operative mechanisms with NEA's Committee on Nuclear Regulatory Activities (CNRA), responsible for the activities of the Agency concerning the regulation, licensing and inspection of nuclear installations with regard to safety. It also co-operates with NEA's Committee on Radiation Protection and Public Health and NEA's Radioactive Waste Management Committee on matters of common interest.

Foreword

The Committee on the Safety of Nuclear Installations (CSNI) of the OECD-NEA co-ordinates the NEA activities concerning the technical aspects of design, construction and operation of nuclear installations insofar as they affect the safety of such installations. In 1994, the CSNI approved a proposal to set up a Task Group under its Principal Working Group 3 (recently re-named as the Working Group on Integrity of Components and Structures (IAGE)) to study the need for a programme of international activities in the area of concrete structural integrity and ageing and how such a programme could be organised. The task group reviewed national and international activities in the area of ageing of nuclear power plant concrete structures and the relevant activities of other international agencies. A proposal for a CSNI programme of workshops was developed to address specific technical issues which were prioritised by OECD-NEA task group into three levels of priority:

First Priority

- Loss of prestressing force in tendons of post-tensioned concrete structures
- In-service inspection techniques for reinforced concrete structures having thick sections and areas not directly accessible for inspection

Second Priority

- Viability of development of a performance based database
- Response of degraded structures (including finite element analysis techniques)

Third Priority

- Instrumentation and monitoring
- Repair methods
- Criteria for condition assessment

The working group has progressively worked through the priority list developed during the preliminary study carried out by the Task Group. Currently almost all of the three levels of priority are effectively complete, although in doing so the committee has identified other specific items worthy of consideration. By working logically through the list of priorities the committee has maintained a clarity of purpose which has been important in maintaining efficiency and achieving its objectives. The performance of the group has been enhanced by the involvement of regulators, operators and technical specialists in both the work of the committee and its technical workshops and by liaison and co-operation with complementary committees of other international organisations. The workshop format that has been adopted (based around presentation of pre-prepared papers or reports followed by open discussion and round-table development of recommendations) has proved to be an efficient mechanism for the identification of best practice, potential shortcomings of current methods and identification of future requirements.

SUMMARY

OECD-NEA workshop on the evaluation of defects, repair criteria & methods of repair for concrete structures on nuclear power plants

OECD-NEA IAGE held an international workshop on the evaluation of defects, repair criteria & methods of repair for concrete structures on nuclear power plants in Berlin, Germany on April 10-11, 2002. Through 2 technical sessions devoted to Operational Experience and State of the Art and Future Developments, a broad picture of the status was given to a large audience composed by 54 participants from 17 countries and International Organisations. 21 papers have been presented at the Workshop.

The objectives of the workshop were to examine the current practices and the state of the art with regard to the evaluation of defects, repair criteria and methods of repair for concrete structures on Nuclear Power Plants with a view to determining the best practices and identification of shortfalls in the current methods, which are presented in the form of conclusions and recommendations in this report.

This workshop on the evaluation of defects, repair criteria and methods of repair for concrete structures on Nuclear Power Plants is the latest in a series of workshops.

The complete list of CSNI reports, and the text of reports from 1993 on, is available on http://www.nea.fr/html/nsd/docs/

Acknowledgement

Gratitude is expressed to GRS, Germany for hosting the Workshop at the DIN Institute in Berlin. In particular, special thanks to Mr. Helmut Schulz and Dr Jurgen Sievers, and also Mrs Brunhilde Laue and Mrs Schneider for their help.

Thanks are also expressed to chairmen of the sessions and to the Organizing Committee for their effort and co-operation.

Dr Leslie M Smith	**BEG(UK) Ltd**	**(UK) Chairman**
Prof Pierre Labbé	IAEA	(International)
M. Jean-Pierre Touret	EdF	(France)
Herr Rüdiger Danisch	Framatome ANP GmbH	(Germany)
Mr James Costello	USNRC	(USA)
Dr Dan Naus	ORNL	(USA)
M.Eric Mathet	OECD-NEA	(International)

OECD-NEA WORKSHOP ON THE EVALUATION OF DEFECTS, REPAIR CRITERIA & METHODS OF REPAIR FOR CONCRETE STRUCTURES ON NUCLEAR POWER PLANTS

10th and 11th April, 2002
Berlin, Germany

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C. PROGRAMME
D. PAPERS
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OECD-NEA Workshop on the Evaluation of Defects, Repair Criteria & Methods of Repair for Concrete Structures on Nuclear Power Plants, GRS, Berlin, Germany April 10-11, 2002

Conclusions and Recommendations

The objectives of the workshop were to examine the current practices and the state of the art with regard to the evaluation of defects, repair criteria and methods of repair for concrete structures on Nuclear Power Plants with a view to determining the best practices and identification of shortfalls in the current methods, which are presented in the form of conclusions and recommendations.

CONCLUSIONS

1. Repairs to concrete NPP structures and their durability will continue to be an issue until final decommissioning.

2. Experience gained during repair projects, and extensive field studies of repaired structures, show that the effectiveness of the concrete repair is dependent on:
   - correct diagnosis of the cause of the damage;
   - selection of a repair strategy that addresses this cause;
   - choice of appropriate repair materials and methods;
   - careful management of the process;
   - post repair maintenance strategy supported by comprehensive records.

Computerised databases can assist with: recording the detection and diagnosis of damage; rexcording the location of, and specification for, repairs; and management of the subsequent repair.

3. The combination of concrete with composite materials is useful in a repair situation. These materials now have a track record in structural repairs to a decommissioned prestressed concrete containment (PCC). Extensive testing has proved their potential as an alternative to steel as a liner for PCCs. They are currently being considered for enhancing the leak tightness of unlined containments.

4. Surface overcoating materials can protect exposed concrete surfaces from deterioration due to environmental factors eg carbonation, chlorides etc. Careful design will ensure that the coating system can accommodate structural movement, maximise durability and satisfy aesthetic considerations.

5. Experiences of repairs, supported by field studies of repaired structures, confirm that a principal cause of damage to reinforced concrete structures is corrosion of the reinforcement. Impressed current cathodic protection (CP) has been shown to be effective in improving the durability of a repaired structure exposed to a very severe marine environment. Laboratory examination of samples of concrete removed from a structure protected by CP has shown that long term application of impressed current CP was not detrimental to the original concrete and did not affect the steel to concrete bond.

6. There was recognition that the nuclear industry might benefit from improved guidance on assessment of defects and the effectiveness of subsequent repairs. However, absolute criteria are difficult to define and may not be universally applicable.

7. Laboratory trials of impact echo and synthetic aperture focusing technique ultrasonic non-destructive testing have shown that they have potential to detect subsurface features in concrete elements but that significant further development is required for field implementation.

8. Laboratory tests are providing important data on the leakage of air and air/steam through cracked concrete. These tests will help to inform the assessment of pressure retaining concrete structures on NPPs and provide a useful source of validation for numerical models and simulation.

9. Papers on the application of structural monitoring to NPP concrete structures confirmed that the practicality of installing instrumentation is of equal importance to its ability to measure the damage parameter under investigation. Acoustic monitoring has demonstrated the potential to detect and locate cracking and may warrant further consideration as a tool for assisting the testing of containment structures.

10. There is little data available on the effect of irradiation on concrete. Samples of concrete removed from a biological shield structure have provided some information.

11. A pre-prepared structural condition assessment procedure listing nuclear safety related structures may be useful in assessing post-fire damage on NPPs. Materials used in the repair of fire damage must be capable of meeting the fire performance criteria required by the original structure.

RECOMMENDATIONS

The following recommendations are offered to inform national activities and research programmes for the inspection, maintenance and repair of concrete NPP structures.

1. The execution and durability of repairs to concrete should be considered as an issue that is relevant to the nuclear safety of NPPs throughout the lifetime of the plant and until final decommissioning.

2. Improved guidance is required on the assessment of defects (eg. cracks) and the performance and effectiveness of subsequent repairs.

3. Owners/operators of NPP concrete structures should develop procedures for recording: the detection and diagnosis of defects/damage; the location of and specification for each repair; and the management strategy to be applied to the repair.

4. Further development of NDE techniques is required in order to support the assessment and evaluation of defects and subsequent repairs in concrete structures. The development priorities, conclusions and recommendations identified at the OECD-NEA 1997 Risley NDE workshop should be applied.

5. Further work is required on the evaluation of leakage through cracks in concrete structures.

6. Further investigation of the effects of irradiation on concrete is required.

7. Consideration should be given to the development of pre-prepared structural condition assessment procedures listing nuclear safety related structures for the evaluation of post-fire damage on NPPs.

OECD-NEA WORKSHOP ON THE EVALUATION OF DEFECTS, REPAIR CRITERIA & METHODS OF REPAIR FOR CONCRETE STRUCTURES ON NUCLEAR POWER PLANTS

Hosted by GRS at the DIN Institute in Berlin, Germany
10 & 11 APRIL 2002

C. PROGRAMME

Wednesday April 10 , 2002

9:30	**10:00**	Welcome	*H. Schulz, GRS* *E. Mathet* - *OECD*
		Introduction	*L. Smith*- *Chairman*

Introductory paper

10:00	**10:30**	Inspection, Assessment and Repair of Nuclear Power Plant Concrete Structures	*D. J. Naus, Oak Ridge National Laboratory, U.S.A.* H. L. Graves, J. F. Costello, USNRC, U.S.A.
10:30	***10:50***	Coffee break	

SESSION A: Operational Experience
Chairman: Mr. Ruediger.Danisch, FRAMATOME-ANP GmbH (GE)

10:50	**11:10**	Repair of the Gentilly-1 Concrete Containment Structure	*A. Popovic*, D. Panesar and M. Elgohary, AECL, (CDN)
11:10	**11:30**	The Repair of Nuclear Power Plant Reinforced Concrete Marine Structures and Installation of an Automated Cathodic Protection System	L. M. *Smith*, C.A. Hughes, British Energy Generation UK ,Ltd. G. Jones, Sea-Probe, Ltd.

11:30	**11:50**	Feasibility Study of IE-SASW Method for the Non-Destructive Evaluation of Containment Building of Nuclear Power Plant	*Mr. Yong-Pyo Suh,* KEPRI (K)
11:50	**12:10**	Field Studies of Effectiveness of Concrete Repairs	*N.J.R. Baldwin*, Mott MacDonald Ltd.,*(UK)*
12:10	**12:45**	Detection and repair of defects in the confinement structures at Paks NPP	*Mr. Nyaradi Csaba, Paks NPP Ltd*
12:45	**14:00**	Lunch	

SESSION A: Operational Experience (Continued)
Chairman: **Dr James Costello, USNRC (USA)**

14:00	**14:20**	Steam Generator Replacement at Ringhals 3 Containment, Transport Opening	*Jan Gustavsson*, Ringhals Nuclear Power Plant, (SW)
14:20	**14:40**	In Service Inspection Programme and Long Time Monitoring of Temelin NPP Containment Structures	Jan Maly, *Jan Stepan*, Energoprojekt Prague, Czech Republic
14:40	**15:00**	Repair Criteria and Methods of Repair for Concrete Structures on Nuclear Power Plants	R. *Lasudry*, Tractebel Energy Engineering, (BE)
15:00	**15:20**	Post-Fire Damage Assessment Procedures for Nuclear Power Plant Structures	L.M. *Smith*, British Energy Generation UK ,Ltd., (UK)
15:20	***15:50***	Coffee break	

SESSION B: State of the Art & Future Developments

		Chairman:	**Dr Naus, ORNL (US)**
15:50	**16:10**	Various stages to address Concrete Cracking on NPPs	*C. Seni,* Mattec Engineering Ltd., (CDN*)*
16:10	**16:30**	Investigation of the Leakage Behaviour of Reinforced Concrete Walls	Nico Herrmann, *Christoph Niklasch*, Michael Stegemann, Lothar Stempniewski, University of Karlsruhe, (GE)
16:30	**16:50**	The Development of a State-of-the-Art Structural Monitoring Instrumentation System for Nuclear Power Plant Concrete Structures	*L. M. Smith*, B. Stafford, M.W. Roberts, British Energy Generation UK ,Ltd. A. McGown, University of Strathclyde *(UK)*
16:50	**17:10**	Ageing and Static Reliability of Concrete Structures under Temperature and Force Loading *Paper not presented but in the proceedings*	Petr Stepanek,, *Stanislav Stastnik* Vlastislav Salajka, Technical University of Brno, Jaroslav Skolai, Jiri Stastny, Dukovany Power Plant, (CZ)
17:10	**17:30**	Efficient management of inspection and monitoring data for a better maintenance of infrastructure	Marcel de Wit, *Gilles Hovhanessian,* Advitam
17:30	**17:45**	Aging process of a good concrete during forty years	*Dr. Peter Lenkei,* University, College of Engineering (Hungary)

End of the first day

Thursday April 11, 2002

SESSION B: State of the Art & Future Developments (Continued)

		Chairman:	**Mr. Jean-Pierre TOURET, EdF, (F)**
9:00	**9:30**	Acoustic monitoring	Marcel de Wit, *Gilles Hovhanessian,* Advitam

9:30	**9:50**	Concrete Properties Influenced by Radiation Dose During Reactor Operation	*Takaaki Konno,* Secretariat of Nuclear Safety Commission, (J)
9:50	**10:10**	The Use of Composite Materials in the Prevention and Strengthening Of Nuclear Concrete Structures	D. Chauvel, P.A. Naze, J-P. *Touret* EdF, Villeurbanne, (F)
10:10	**10:30**	Detection of reinforcement corrosion and its use for service life assessment of concrete structures	C. Andrade, *I . Martínez,* J. Muñoz, CSIC (SP) Rodríguez, M. Ramírez, GEOCISA (SP)
10:30	***11:00***	Coffee break	
11:00	**11:20**	Improved Detection of Tendon Ducts and Defects in Concrete Structures Using Ultrasonic Imaging	W. Müller, V. *Schmitz,* FIZP, (GE) M. Krause, M. Wiggenhauser, Bundesanstalt für Materialforschung und - Prüfung, (GE)
11:20	**11:40**	Structural Integrity Evaluation of a Steel Containment for the Replacement of Steam Generator	*Mr. Yong-Pyo Suh,* KEPRI (KR)
11:40	**12:00**	New Methods on Reconstruction of Safety Compartments of Nuclear Power Plants	Z. Köpper, Köpper und Partner, Bochum, (GE) *D. Busch,* RWE Solutions AG, Essen, (GE)
12:00	**14:00**	Lunch	

SESSION C

		Chairman:	**Dr L.M. Smith (UK)**
14:00	**16:30**	**Panel discussion**	
16:30		Closure	

C. PAPERS

INSPECTION, ASSESSMENT, AND REPAIR OF NUCLEAR POWER PLANT CONCRETE STRUCTURES

D.J. Naus
Oak Ridge National Laboratory (ORNL)
Oak Ridge, TN

H.L. Graves, III and J.F. Costello
U.S. Nuclear Regulatory Commission (USNRC)
Washington, D.C.

Abstract

Aging of concrete structures occurs with the passage of time and has the potential, if its effects are not controlled, to increase the risk to public health and safety. Activities have been conducted to address factors related to quantifying the effects of age-related degradation on nuclear power plant concrete structures and components. Environmental effects that can lead to age-related degradation of reinforced concrete structures and their manifestations are described. Current regulatory in-service testing and inspection requirements are reviewed. Techniques commonly used to inspect nuclear power plant concrete structures to assess and quantify age-related degradation are identified. An approach for conduct of condition assessments is presented, as well as criteria, based on visual indications, for use in classification and assessment of concrete degradation. Materials and techniques for repair of reinforced concrete structures are noted and guidance provided on repair options available for various forms of concrete degradation (e.g., cracking, spalling, and steel reinforcement corrosion). Nuclear power plant degradation and repair experience is summarized. In-service inspection/repair strategies to maintain the probability of failure of a concrete component at or below a target value are discussed.

1. Introduction

To date, 104 nuclear power reactors are currently licensed for commercial operation in the United States (US). Currently 103 of these reactors are in operation, producing about 20% of the nation's electricity supply. The median age of these reactors is over 20 years, with 61 having been in commercial operation for 20 or more years. Initial operating licenses for these reactors will start expiring in 2006, with approximately 10% expiring by the year 2010, and more than 40% by the year 2015. Continuing the service of existing nuclear power plants (NPPs) through a renewal of their initial operating licenses provides a timely and cost-effective solution to the problem of meeting future electricity demand. In fact, 48 reactor units (as of March 2002) have either completed the license renewal process, submitted applications to renew their operating licenses, or announced that they intend to do so. However, the structures in these plants are susceptible to aging by various processes, depending on the operating environment and service conditions, that can affect the engineering properties, structural resistance/capacity, failure mode, and location of failure initiation. As a result, the ability of the structures to withstand various challenges in service from operating conditions, the natural environment, and accidents may be impacted. Current aging-related activities in large measure are therefore focusing technical development and support on condition assessment with the aim of demonstrating that structural margins of existing plants have not or will not erode during the desired service life due to aging or environmental effects. Probabilistic methods can be used to provide the quantitative tools for the assessment of uncertainty in condition assessment and are an essential ingredient of risk-informed management decisions concerning continued service of the NPP structures.

2. NPP Concrete Containments

All commercial NPPs contain concrete structures whose performance and function are necessary for protection of the safety of plant operating personnel and the general public. Most of the concrete structures in NPPs are similar to conventional civil engineering structures; however, certain NPP concrete structures can entail thicker sections, increased reinforcement, and limited accessibility and harsher exposure conditions at certain locations. Typical safety-related functions that the concrete structures provide include foundation, support, shielding, and containment. Although a number of concrete structures are important to the overall safety of NPPs (e.g., fuel/storage pools, cooling water intake structures, and foundations), discussions will concentrate on the containment structures because of their unique requirements.

Concrete containments are metal lined, reinforced concrete pressure-retaining structures that in some cases may be post-tensioned. The concrete vessel includes the concrete shell and shell components, shell metallic liners, and penetration liners that extend the containment liner through the surrounding shell concrete. The reinforced concrete shell, which generally consists of a cylindrical wall with a hemispherical or ellipsoidal dome and flat base slab, provides the necessary structural support and resistance to pressure-induced forces. Leak-tightness is provided by a steel liner fabricated from relatively thin plate material (e.g., 6-mm thick) that is anchored to the concrete shell by studs, structural steel shapes, or other steel products. Initially, existing building codes, such as American Concrete Institute (ACI) Standard 318, *Building Code Requirements for Reinforced Concrete* [1], were used in the nuclear industry as the basis for design and construction of concrete structural members. However, because the existing building codes did not cover the entire spectrum of design requirements and because they were not always considered adequate, additional criteria were developed for design of seismic Category 1 (i.e., safety related) structures (e.g., definitions of load combinations for both operating and accident conditions). Plants that used early ACI codes for design were reviewed by the USNRC through the Systematic Evaluation Program to determine if there were any unresolved safety concerns [2]. Current rules for construction of concrete containments are provided in Section III, Division 2 of the ASME Code [3]. The USNRC has developed supplemental load combination criteria and provides information related to concrete and steel internal structures of steel and concrete containments [4,5]. Rules for design and construction of the metal liner that forms the pressure boundary for the reinforced concrete containments are found in Section III, Division 1, Subsection NE of the ASME Code. Depending on the functional design (e.g., large dry or ice condenser), NPP concrete containments can be on the order of 40 to 50 m diameter and 60 to 70 m high, with wall and dome thicknesses from 0.9 to 1.4 m, and base slab thicknesses from 2.7 to 4.1 m. Almost three-quarters of the NPPs licensed for commercial operation in the US employ either a reinforced concrete or post-tensioned concrete containment. Boiling-water reactor plants in the US that utilize a steel containment have reinforced concrete structures that serve as secondary containments or reactor buildings that provide support and shielding functions for the primary containment.

3. Potential Degradation Factors

Degradation is considered to be any phenomenon that decreases the load-carrying capacity of the containment, limits its ability to contain a fluid medium, or reduces its service life. Service-related degradation can affect the ability of a NPP containment to perform satisfactorily in the unlikely event of a severe accident. The root cause for component degradation can generally be linked to a design or construction problem, inappropriate material application, a base- or weld-metal flaw, maintenance or inspection activities, or a severe service condition. Primary mechanisms or factors that can produce premature deterioration of concrete structures include those that impact either the concrete or reinforcing steel materials (i.e., mild steel reinforcement or post-tensioning system). Degradation of concrete can be caused by adverse performance of its cement-paste matrix or aggregate materials under either chemical or physical attack. Chemical attack may occur in several forms: efflorescence or leaching, sulfate attack (including delayed ettringite formation), attack by acids and bases, salt crystallization, and alkali-aggregate reactions. Physical attack mechanisms for concrete include freeze/thaw cycling, thermal exposure/thermal cycling, abrasion/erosion/cavitation, irradiation, and fatigue or vibration. Degradation of mild steel reinforcing materials can occur as a result of corrosion, irradiation, elevated temperature, or fatigue effects. Post-tensioning systems are susceptible to the same degradation mechanisms as mild steel reinforcement, plus loss of prestressing force, primarily due to tendon relaxation and concrete creep and shrinkage.

4. Testing and Inspection Requirements

One of the conditions of all operating licenses for water-cooled power reactors is that the primary reactor containment shall meet the requirements set forth in Appendix J, "Primary Reactor Containment Leakage Testing for Water-Cooled Power Reactors," to 10 CFR Part 50 [6]. Contained in Appendix J are requirements pertaining to Type A, B, and C leakage-rate tests that must be performed by each licensee as a condition of their operating license. On September 26, 1995, the USNRC amended Appendix J (60 FR 49495) to provide a performance-based option for leakage-rate testing as an alternative to the existing prescriptive requirements. The amendment is aimed at improving the focus of the body of regulations by eliminating prescriptive requirements that are marginal to safety and by providing licensees greater flexibility for cost-effective implementation methods for regulatory safety objectives.

Appendix J to 10 CFR Part 50 requires a general inspection of the accessible interior and exterior surfaces of the containment structures and components to uncover any evidence of structural deterioration that may affect either the containment structural integrity or leak-tightness. On August 8, 1996, the USNRC published an amendment (61 FR 41303) to 10 CFR 50.55a of its regulations to require that licensees use portions of the ASME Code for containment in-service inspection. Specifically, the rule requires that licensees adopt the 1992 Edition with the 1992 Addenda of Subsection IWE, "Requirements for Class MC and Metallic Liners of Class CC Components of Light-Water Cooled Power Plants," and Subsection IWL, "Requirements for Class CC Concrete Components of Light-Water Cooled Power Plants," of Section XI. In addition, several supplemental requirements with respect to the concrete and metal containments were included in the rule. On September 22, 1999 the USNRC again amended 10 CFR Part 50.55a to endorse use of the 1995 Edition up to and including 1996 Addenda of Section XI, Subsections IWE and IWL, of the ASME Code for inspection of containment structures. Subsequently on August 3, 2001, the USNRC announced that it intends to amend 10 CFR Part 50.55a to incorporate by reference the 1997 Addenda, the 1998 Edition, the 1999 Addenda, and the 2000 Addenda of Section XI of the ASME Code [7]. Comments on the proposed amendment are presently being addressed.

5. In-Service Inspection and Condition Assessment

Operating experience has demonstrated that periodic inspection, maintenance, and repair are essential elements of an overall program to maintain an acceptable level of reliability over the service life of a NPP containment, or in fact, of any structural system. Knowledge gained from conduct of an in-service condition assessment can serve as a baseline for evaluating the safety significance of any degradation that may be present, and defining subsequent in-service inspection programs and maintenance strategies.

Effective in-service condition assessment of a containment requires knowledge of the expected type of degradation, where it can be expected to occur, and application of appropriate methods for detecting and characterizing the degradation. Degradation detection is the first and most important step in the condition assessment process. Routine observation, general visual inspections, leakage-rate tests, and nondestructive examinations are techniques used to identify areas of the containment that have experienced degradation. Techniques for establishing time-dependent change such as section thinning due to corrosion, or changes in component geometry and material properties, involve monitoring or periodic examination and testing.

Knowing where to inspect and what type of degradation to anticipate often requires information about the design features of the containment as well as the materials of construction and environmental factors. Basic components of the continued service evaluation process for NPP concrete structures include damage detection and classification, root-cause determination, and measurement.

5.1 In-service inspection

Nondestructive test methods are used to determine hardened concrete properties and to evaluate the condition of concrete in structures.[1] Application of these methods for detection of degradation in reinforced concrete structures involves either a direct or indirect approach. The direct approach generally involves a visual inspection of the structure, removal/testing/analysis of material, or a combination of the above. Indirect approaches measure some property of concrete (e.g., rebound number or ultrasonic pulse velocity) and relate it to strength, elastic behavior, or extent of degradation through correlations that have been established previously. Many of the nondestructive test methods are based on the indirect approach, in which a small number of destructive and nondestructive tests are conducted in tandem at noncritical locations in a structure to develop the required correlation curve(s). However, destructive tests may not be possible in many areas of a NPP structure to develop the required curves so assessment of in-place strength must be based on published relations. Environment-specific methods are used where surfaces of structures are not accessible for direct inspection due to the presence of soils, protective coatings, or portions of adjacent structures. These methods provide an indirect assessment of the physical condition of the structure (i.e., potential for degradation) by quantifying the aggressiveness of the environment adjacent to the structure (e.g., air, soil, and groundwater). If results of these tests indicate that the environment adjacent to the structure is not aggressive, one might conclude that the structure is not deteriorating. However, when conditions indicate that the environment is potentially conducive to degradation, additional assessments are required that may include exposure of the structure for visual or limited destructive testing.

5.2 Condition assessment

Determining the existing performance characteristics and extent and causes of any observed distress is accomplished through a condition assessment. Common in the condition assessment approaches is the conduct of a field survey, involving visual examination and application of nondestructive and destructive testing techniques, followed by laboratory and office studies. Guidelines and direction on conduct of surveys of existing general civil engineering buildings are available [10,11]. The condition survey usually begins with a review of the "as-built" drawings and other information pertaining to the original design and construction so that information, such as accessibility and the position and orientation of embedded steel reinforcing and plates in the concrete, is known prior to the site visit. Next is a detailed visual examination of the structure to document easily obtained information on instances that can result from or lead to structural distress. Visual inspections are one of the most valuable of the condition survey methods because many of the manifestations of concrete deterioration appear as visible indications or discontinuities on exposed concrete surfaces. Visual inspections encompass a variety of techniques (e.g., direct and indirect inspection of exposed surfaces, crack and discontinuity mapping, physical dimensioning, environmental surveying, and protective coatings review). To be most effective, the visual inspection should include all exposed surfaces of the structure; joints and joint materials; interfacing structures and materials (e.g., abutting soil); embedments; and attached components (e.g., base plates and anchor bolts). Degraded areas of significance are measured. The condition of the surrounding structures should also be examined to detect occurrence of differential settlement or note aggressiveness of the local operating environment. Results obtained should be documented and photographs or video images taken of

[1] Descriptions and principles of operation, as well as applications, for nondestructive test methods most commonly used to determine material properties of hardened concrete in existing construction and to determine structural properties and assess conditions of concrete are available [8,9].

any discontinuities and pertinent findings. A crack survey is usually done by drawing the locations and widths of cracks on copies of project plans. Cracking patterns may appear that suggest weaknesses in the original design, construction deficiencies, unanticipated thermal movements, chemical reactivity, detrimental environmental exposure, restrained drying shrinkage, or overloading. Distress associated with cracks such as efflorescence, rust stains, or spalling is noted. After the visual survey has been completed, the need for additional surveys such as delamination plane, corrosion, or pachometer is determined. The delamination plane survey is used to identify internal delaminations that are usually caused by corrosion of embedded metals or internal vapor pressure. Results of the visual and delamination surveys are used to select portions of the structure that will be studied in greater detail. To locate areas of corrosion activity within reinforced concrete, copper-copper sulfate half-cell studies can be performed. By taking readings at multiple locations on the concrete surface, an evaluation of the probability of corrosion activity of embedded reinforcing steel (or other metals) can be made. Where significant chloride penetration is suspected, concrete powder samples or cores should be removed from several depths extending to and beyond the embedded outer layer of reinforcing steel. Also, a pachometer survey may be performed as part of the detailed study to confirm the location of steel reinforcement. Where there is evidence of severe corrosion, the steel bar should be uncovered to allow visual inspection and measurement of cross-sectional area loss. Upon return to the office, results of the field survey are evaluated in detail. A crack survey map is prepared and studied for meaningful patterns. Half-cell data are studied and isopotential lines are drawn to assist in determining active corrosion sites. Samples of concrete and steel obtained from areas exhibiting distress are tested in the laboratory. Chloride ion results are plotted versus depth to determine the profile and the chloride content at the level of the steel. Any elements that appear to be structurally marginal, due either to unconservative design or deterioration effects, are identified and appropriate calculation checks made. These analyses may identify distress in the structure that has been caused by structural overload and indicate safety factors. If the calculations are inconclusive, suitable load testing may be indicated (if feasible). After all of the field and laboratory results have been collated and studied and all calculations have been completed, a report is prepared.

Cracking is a very common damage by-product from a large number of concrete degradation mechanisms. Active concrete cracking is difficult to assess in terms of impact on structural behavior and is difficult to repair. Thus, inspection methods that support the early identification, sizing, and cause of cracking in concrete structures are of primary interest for future inspections. Also, the primary concern for all metallic constituents of concrete structures is corrosion and corrosion-related damage. Inspections that identify early signs of corrosion cell initiation and indicate the rate of propagation are similarly valuable. A visual-based approach based primarily on the results of visual inspections has been developed for assistance in the classification and treatment of conditions or findings that might emanate from in-service inspections of reinforced concrete structures.•

The visual-based approach uses a "three-tiered" hierarchy" so that through use of different levels of acceptance, minor discontinuities can be accepted and more significant degradation in the form of defects can be evaluated in more detail [13,14]. The three acceptance levels include acceptance without further evaluation, acceptance after review, and additional evaluation required. Criteria associated with these acceptance levels are presented elsewhere [13]. Evaluations under these acceptance levels can involve extensive application of both nondestructive and destructive testing methods and detailed analytical evaluations frequently may be required to better characterize the current condition of the structure and provide the basis for formulation of a repair strategy (if needed). Even if the analysis results indicate that the component is acceptable at present, additional assessments should be conducted to demonstrate that the component will continue to meet its functional and performance requirements during the desired service

• Information is also available on a damage-based approach that is founded on the concept that degradation of a component in service is manifested in physical evidence (i.e., measurable values) that can be categorized or classified into distinct stages or conditions in accordance with their impact on performance [12].

life (i.e., take into account the current structural condition and use service life models to estimate the future impact of pertinent degradation factors on performance).

6. Repair of Reinforced Concrete

Reinforced concrete structures can start to deteriorate due to exposure to the environment (e.g., temperature, moisture, and cyclic loading) almost from the time of construction [15]. The rate of deterioration is dependent on the component's structural design, materials selection, quality of construction, curing, and aggressiveness of its environmental exposure. Termination of a component's service life occurs when it can no longer meet its functional and structural requirements. Results provided through periodic application of in-service inspection techniques as part of a condition assessment program can be used to develop and implement a remedial action prior to the structure achieving an unacceptable level of performance. Depending on the degree of deterioration and the residual strength of the structure, the function of a remedial measures activity may be structural, protective, cosmetic, or any combination of these.

6.1 Repair considerations

The first step in any repair activity is a thorough assessment of the damaged structure or component including evaluation of the (1) cause of deterioration, (2) extent of deterioration, and (3) effect of deterioration on the functional and performance requirements of the structure or component. From this information a remedial measures strategy is developed based on the consequence of damage (e.g., affect of degradation on structural safety), time requirements for implementation (e.g., shutdown requirements, immediate or future safety concern), economic aspects (e.g., partial or complete repair), and residual service life requirements (e.g., desired residual service life will influence action taken) [16]. Basic remedial measures options include (1) no active intervention; (2) more frequent inspections or conduct of specific studies; (3) if safety margins are presently acceptable, take action to prevent deterioration from getting worse; (4) carry out repairs to restore deteriorated or damaged parts of structure to a satisfactory condition; and (5) demolish and rebuild all or part of structure. Quite often options (3) and (4) are considered jointly.

6.2 Repair materials and techniques

Deterioration of reinforced concrete generally will result in cracking, spalling, or delamination of the cover concrete. Corrosion resulting from either carbonation or the presence of chlorides is the dominant type of distress that impacts reinforced concrete structures. More detailed information to that provided below on typical remedial measures for NPP concrete structures is available [16-18].

After identifying that a crack is of sufficient size to require repair, it is important to determine if the crack is dormant or active (i.e., mechanism still operating). Dormant cracks can be resin injected using epoxy or high molecular weight methacrylate (HMWM). Active cracks must be treated as if they are control joints and require special treatment, especially if fluid leakage is involved. Surface preparation is critical to a successful spall repair. The concrete substrate must be sound and the exposed surface dry and free of oil, grease, and loose particles. The most appropriate materials for patching are those that are closest in composition to the material to be patched. Usually this means portland cement concrete for large patches or portland cement mortar for small ones; however, non-portland cement binders have been used successfully. By patching with a cementitious material, the final thermal and structural properties of the repair will be similar to the base concrete. Where the repairs are exposed to aggressive fluids the chemical composition of the fluids should be known and the repair materials must be compatible. Delaminations can be repaired by removal and replacement of the delaminated concrete. In areas where removal of concrete is not required, the delaminated area can be repaired by injection of epoxy or HMWM. Proper surface preparation, batching, mixing, placing, and curing are all important for long-term durability of concrete repairs. Basic repair solutions for corrosion-damaged reinforced concrete include: (1) realkalization by either direct replacement of contaminated concrete with new concrete, use of a cementitious material overlay, or application of electrochemical means to accelerate diffusion of alkalis into carbonated concrete; (2) limiting the corrosion rate by changing the environment (e.g., drying) to reduce the electrolytic conductivity; (3) steel reinforcement coating (e.g., epoxy); (4) chloride extraction by passing an electric current (DC) from an anode attached to the concrete surface through the concrete to the reinforcement (chloride ions migrate to anode); and (5) cathodic protection.

6.3 NPP repair experience

A survey was distributed to solicit information on the locations and types of concrete distress commonly found in US NPPs [17].• Twenty-nine plants representing forty-two reactor units responded. The results of this survey are summarized below:

- Concrete structure evaluations are usually limited to an assessment of prestressing systems of post-tensioned concrete containments and a general visual survey of exposed concrete surfaces;
- Twenty-six of the plants reported concrete damage or deterioration with cracking and spalling being most common;
- Most common locations of deterioration in BWR plants were in the containment dome and in the walls and slabs of auxiliary structures, and in PWR plants the locations were in slabs, walls and equipment supports of reactor buildings and auxiliary structures;
- Twenty-seven of the plants have repaired damaged concrete with epoxy injection, grout injection, and flexible sealing of cracks being the most common methods utilized; and
- Follow-up evaluation of concrete repairs were not commonly performed.

In general, many of the reported degradation instances associated with the NPP concrete structures occurred early in the life of the structures and have been attributed to construction/design deficiencies, improper material selection, or environmental effects. Examples of some of the specific problems that have occurred due to age-related degradation include concrete containment liner corrosion, leaching of tendon gallery concrete, corrosion of steel reinforcement in water-intake structures, failure of prestressing tendon wires due to corrosion, and freeze-thaw damage to containment dome. Although the vast majority of the structures will continue to meet their functional and performance requirements during their service period, it is reasonable to assume that there will be isolated incidents where the structures may not exhibit the desired durability without some form of intervention.

7. Time-Dependent Reliability

Evaluation of structures for continued service should provide quantitative evidence that their capacity is sufficient to withstand future demands within the proposed service period with a level of reliability sufficient for public safety. Structural aging will cause the integrity of structures to evolve over time (e.g., a hostile service environment may cause structural strength and stiffness to degrade). Uncertainties that complicate the evaluation of aging effects arise from a number of sources: inherent

• This survey was conducted prior to the ammendment to 10 CFR 50.55a requiring licensees to use Subsections IWE and IWL of the ASME Code for containment in-service inspections, and conduct of inspections of selected plants by the USNRC [19].

randomness in structural loads, initial strength, and degradation mechanisms; lack of in-service inspection measurements and records; limitations in available databases and models for quantifying time-dependent material changes and their contribution to structural capacity; inadequacies in non-destructive evaluation; and shortcomings in existing methods to account for repair. Any evaluation of the reliability of a reinforced concrete structure during its service life must take into account these effects, plus any previous challenges to the integrity that may have occurred.

Time-dependent reliability analysis methods provide a framework for performing condition assessments of existing structures and for determining whether in-service inspection and maintenance are required to maintain reliability and performance at the desired level. The duration of structural loads that arise from rare operating or environmental events, such as accidental impact, earthquakes, and tornadoes, is short and such events occupy a negligible fraction of a structure's service life. Such loads can be modeled as a sequence of short-duration load pulses occurring randomly in time. The occurrence in time of such loads is described by a Poisson process, with the mean (stationary) rate of occurrence, λ random intensity, S_j, and duration, τ The number of events, N(t), to occur during service life, t, is described by the probability mass function,

$$P\left[N(t) = n\right] = \frac{(\lambda t)^n \cdot \exp(-\lambda t)}{n!}\ ; n = 0,\ 1,\ 2,\ \ldots \tag{1}$$

The intensity of each load is a random variable, described by the cumulative distribution function (CDF) $F_i(x)$. In general, the load process is intermittent and the duration of each load pulse has an exponential distribution,

$$F_{T_d} = 1 - \exp[-t/\tau];\ t \geq 0 \tag{2}$$

in which τ = average duration of the load pulse. The probability that the load process is nonzero at any arbitrary time is $p = \lambda\tau$. Loads due to normal facility operation or climatic variations may be modeled by continuous load processes. A Poisson process with rate λ may be used to model changes in load intensity if the loads are relatively constant for extended periods of time.

The strength, R, of a structural component is described by

$$R = B \bullet R_m(X_1, X_2, \ldots, X_m) \tag{3}$$

in which X_1, X_2 ... are basic random variables that describe yield strength of steel, compressive or tensile strength of concrete, and structural component dimensions or section properties. The function $R_m(\ldots)$ describes the strength based on principles of structural mechanics. Modeling assumptions invariably must be made in deriving $R_m(\ldots)$ and the factor B describes errors introduced by modeling and scaling effects. The probability distribution of B describes bias and uncertainty that are not explained by the model $R_m(\ldots)$ when values of all variables X_i are known. The probability distribution of B can be assumed to be normal. A more accurate behavioral model leads to a decrease in the mean and variability in B and thus in R. Probabilistic models for R usually must be determined from the statistics of the basic variables, X_i, since it seldom is feasible to test a sufficient sample of structural components to determine the cumulative distribution function (CDF) of R directly.

The failure probability of a structural component can be evaluated as a function of (or an interval of) time if the stochastic processes defining the residual strength and the probabilistic characteristics of the loads at any time are known. The strength, R(t), of the structure and applied loads, S(t), are both random functions of time. Assuming that degradation is independent of load history, at any time t the margin of safety, M(t), is

$$M(t) = R(t) - S(t). \tag{4}$$

Making the customary assumption that R and S are statistically independent random variables, the (instantaneous) probability of failure is,

$$P_f(t) = P[M(t) < 0] = \int_0^{\infty} F_R(x)\, f_S(x)\, dx \tag{5}$$

in which $F_R(x)$ and $f_S(x)$ are the CDF of R and probability density function (PDF) of S. Equation (5) provides an instantaneous quantitative measure of structural reliability, provided that $P_f(t)$ can be estimated and/or validated [20]. It does not convey information on how future performance can be inferred from past performance.

For service life prediction and reliability assessment, one is more interested in the probability of satisfactory performance over some period of time, say (0,t), than in the snapshot of the reliability of the structure at a particular time provided by Eq. (5). Indeed, it is difficult to use reliability analysis for engineering decision analysis without having some time period in mind (e.g., an in-service maintenance interval). The probability that a structure survives during interval of time (0,t) is defined by a reliability function, L(0,t). If, for example, n discrete loads S_1, S_2, ..., S_n occur at times t_1, t_2, ..., t_n during (0,t), the reliability function becomes,

$$L(t) = P[R(t_1) > S_1 , \ldots , R(t_n) > S_n] \tag{6}$$

in which $R(t_i)$ = strength at time of loading S_i.

Taking into account the randomness in the number of loads and the times at which they occur as well as initial strength, the reliability function becomes [21]

$$L(t) = \int_0^{\infty} \exp\left(-\lambda t\left[1 - t^{-1}\int_0^t F_s(g_i r)\, dt\right]\right) f_{R_0}(r)\, dt \tag{7}$$

in which f_{R_0} = PDF of the initial strength R_0 and g_i = fraction of initial strength remaining at time of load S_i. The probability of failure during (0,t) is

$$F(t) = 1 - L(t). \tag{8}$$

The conditional probability of failure within time interval (t, t+ Δt), given that the component has survived up to t, is defined by the hazard function which can be expressed as

$$h(t) = -d\, [\ln L(t)]/dt. \tag{9}$$

The reliability and hazard functions are integrally related

$$L(t) = \exp\left[-\int_0^t h(x)dx\right] \tag{10}$$

The hazard function is especially useful in analyzing structural failures due to aging or deterioration. For example, if the structure has survived during the interval (0, t_1), it may be of interest in scheduling in-service inspections to determine the probability that it will fail before t_2. Such an assessment can be performed if h(t) is known. If the time-to-failure is T_f, this probability can be expressed as

$$P\left[T_f < t_2 \middle| T_f > t_1\right] = 1 - \exp\left(-\int_{t_1}^{t_2} h(x)dx\right) \tag{11}$$

In turn, the structural reliability for a succession of inspection periods is

$$L(0,t) = \prod_t L\left(t_{i-1}, t_i\right)\exp\left\{-\int_{t_i}^{t} h(x)dx\right\} \tag{12}$$

in which $t_{i-1} = 0$ when i = 1.

Intervals of inspection and maintenance that may be required as a condition for continued operation can be determined from the time-dependent reliability analysis. Forecasts of reliability enable the analyst to determine the time period beyond which the desired reliability of the structure cannot be assured. At such a time, the structure should be inspected. The density function of strength, based on prior knowledge of the materials in the structure, construction, and standard methods of analysis, is indicated by $f_R(r)$. The information gained during scheduled inspection, maintenance and repair causes the characteristics of strength to change; this is denoted by the (conditional) density $f_R(r|B)$, in which B is an event dependent on in-service inspection. Information gained from the inspection usually involves several structural variables including dimensions, defects, and perhaps an indirect measure of strength or stiffness. If these variables can be related through event B, then the updated density of R following in-service inspection is,

$$f_R\left(r|B\right) = P\left[r < R \leq r + dr, B\right] / P\left[B\right] = c\ K(r)\ f_R(r) \tag{13}$$

in which $f_R(r)$ is termed the prior density of strength, K(r) is denoted the likelihood function, and c is a normalizing constant. The time-dependent reliability analysis then is re-initialized following in-service inspection/repair using the updated $f_R(r|B)$ in place of $f_R(r)$. The updating causes the hazard function to be discontinuous.

Optimal intervals of inspection and repair for maintaining a desired level of reliability can be determined based on minimum life cycle expected cost considerations. Preliminary investigations of such policies have found that they are sensitive to relative costs of inspection, maintenance, and failure [22]. If the cost of failure is an order (or more) of magnitude larger than inspection and maintenance costs, the optimal policy is to inspect at nearly uniform intervals of time. However, additional research is required before such policies can be finalized as part of an aging management plan. Applications of the time-dependent reliability methodology to concrete components are available [22-24].

8. Conclusions

The performance of reinforced concrete structures in NPPs has been good, reflecting the initial quality control, the young age, and the generally benign environment within the a plant. However, as these structures age incidences of degradation are likely to increase and if not controlled, degradation has the potential to reduce the margins that the structures have to withstand various challenges in service from operating conditions, the natural environment, and accidents. The most common form of degradation observed in NPPs has been concrete cracking. When properly used and applied, in-service inspection techniques are effective in detecting aging effects and providing vital input for assessing the condition of structures. Methods for conduct of condition assessments of reinforced concrete structures are fairly well established and generally start with a visual examination of the structure's accessible surfaces. Some guidance has been developed to aid in interpreting results of the condition assessment, but more definitive criteria are required to assist in interpreting the data provided. Repair methods for general civil engineering reinforced concrete structures are fairly well established and effective when properly implemented, however, the long-term effectiveness (or durability) of remedial measures require development. Time-dependent reliability analysis methods provide a framework for performing condition assessments of existing structures and determining whether in-service inspection and maintenance are required to maintain reliability and performance at the desired level, however, quantitative data for input into the methodology are limited and the reliability models for condition assessment have not been validated.

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SESSION A: OPERATIONAL EXPERIENCE

Chairman: Mr. Rüdiger Danisch, Framatome-ANP GmbH (Germany)

REPAIR OF THE GENTILLY-1 CONCRETE CONTAINMENT STRUCTURE

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Abstract

Gentilly-1 CANDU Nuclear Power Plant is located in Gentilly, Quebec, south of the St. Lawrence River. Gentilly-1 was designed in the late 1960'ies and constructed in the mid 1970'ies. The reactor building ring beam has suffered concrete degradation for more than fifteen years and has been repaired in 1985 and in 1993. These repairs were ineffective since extensive deterioration had continued to occur. The ring beam contains and protects the prestressing anchorages and the horizontal ring beam tendons, in addition to the pre-stressing anchorages for the dome prestressing system. In August 1998, an assessment program for the concrete containment building was initiated. The investigation showed that the structural concrete forming the dome and the perimeter wall is expected to endure without ageing problems for at least fifty years. However, portions of the ring beam concrete 'secondary concrete' needed repair. Various design options for the ring beam repair were considered. Atomic Energy of Canada Ltd. (AECL) and Intelligent Sensing for Innovating Structures (ISIS) Canada developed a design for the remedial work and a subsequent monitoring system. The innovative Ring Beam Repair Program was implemented and successfully performed in 2000/2001. The purpose of this paper is to describe the design and field implementation of the repair program.

Introduction

The Plant was designed by AECL in the late 1960'ies and became operational in 1972. In 1980, AECL decided to place the station in a lay-up state and to generate a plan for its final disposition. It was concluded in 1984 that returning the site to a condition of completely unrestricted access and usage was not immediately necessary and, for technical and financial considerations, the attainment of this objective should be delayed for the next fifty to eighty years. AECL, the owner of the facility, was required to maintain the facility in a "static state". In general, the purpose of the maintaining of the facility was to provide interim storage for all conventional and radiological hazards until the facility is finally decommissioned and demolished. Therefore, it was decided that Gentilly-1 containment structure would be required to function for more than fifty years beyond its originally expected design life. However, the structure was visibly aging in the ring beam secondary concrete area that was protecting the prestress anchorages. The poor appearance of the structure was significantly influencing perception of safety and proper functioning of the nuclear installations. The ring beam deterioration is shown in Figures 1a and 1b.

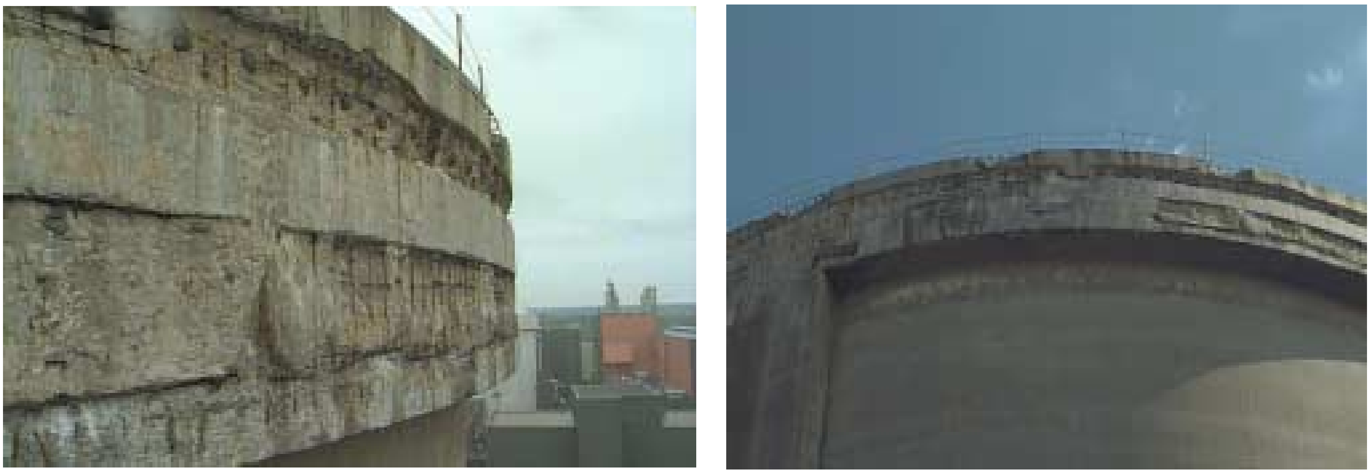


Figure 1a, b: G-1 Ring Beam, South Side

Figure 2: Gentilly-1 Containment Building – Installation of Formwork (April 2000)

A detailed study was undertaken to perform a condition assessment of the structure and the actions necessary to ensure satisfactory performance of the structure over the remaining period of required service life. Structural condition assessment studies have been completed and it has been concluded that, except the part of the secondary concrete in the ring beam, the concrete containment structure has the potential to be serviceable possibly for fifty years or more, as required by AECL.
Based on the findings from the condition assessment, recommendations were developed for the repair design and implementation performed on the ring beam to ensure the satisfactory performance of all parts of the concrete containment structure.

Condition Assessment and Design Requirements

As a result of the condition assessment completed in the year 2000, and all previous investigations, valuable information was gathered based on: visual inspections, in-situ measurements and laboratory testing and analysis. A comprehensive analysis which included testing of concrete, reinforcing steel and prestressing steel materials, chloride ion content, water soluble alkali content, air voids, water absorption, compressive strength, modulus of elasticity, Poisson's ratio, etc., concluded that the containment structure was in good structural condition. It was also concluded that the reactor building could be expected to remain serviceable for the next fifty years if the ring beam repair is performed.

The repair of the ring beam of the Gentilly-1 containment structure was developed by AECL and ISIS Canada. The objective of the concrete repair was to: to remove all unsound and unbonded concrete; to restore the concrete of the ring beam; to protect pre-stressing anchorages; and to improve aesthetics of the building and increase public confidence. The Project had challenges relating to the technical component of the repair, as well as to the logistic construction challenges due to the work at the top of the reactor building, as shown in Figure 2.

The technical part of the concrete repair project had three main tasks:

1. Concrete demolition (removal of unsound concrete) and surface preparation.
2. Concrete repair.
3. Fiber Reinforced Polymers (FRP) protection of the repaired area.

Concrete demolition, surface preparation and concrete repair are part of the conventional engineering tasks and adequate experience and expertise can yield a high standard of repair design and installation.
On the other hand, protection of the repaired concrete using FRP is a newer area in the nuclear industry and does not have an extensive track record. Therefore, a team of specialists was assembled to study and propose the technical solution for the application of FRP.

FRP composites have been used for nearly thirty years in aerospace and manufacturing applications where low weight, high tensile strength and non-corrosive structural properties are required. In civil engineering, applications of different types of FRPs are finding their role in fabric roof structures, internal concrete reinforcement, deck grating and as externally bonded reinforcement or protection. The FRP system has proven benefits in some applications. The technique, known as a wet lay-up, provides flexibility, constructability and short installation times, resulting in lower overall cost. The system may use different type of fibers: carbon, E-glass and Aramid, depending on the particular requirements of an application. The fiber fabric is installed using epoxy resin formulated for substrate adhesion, durability and constructability.

Figure 3 details the application concept of FRP sheets.

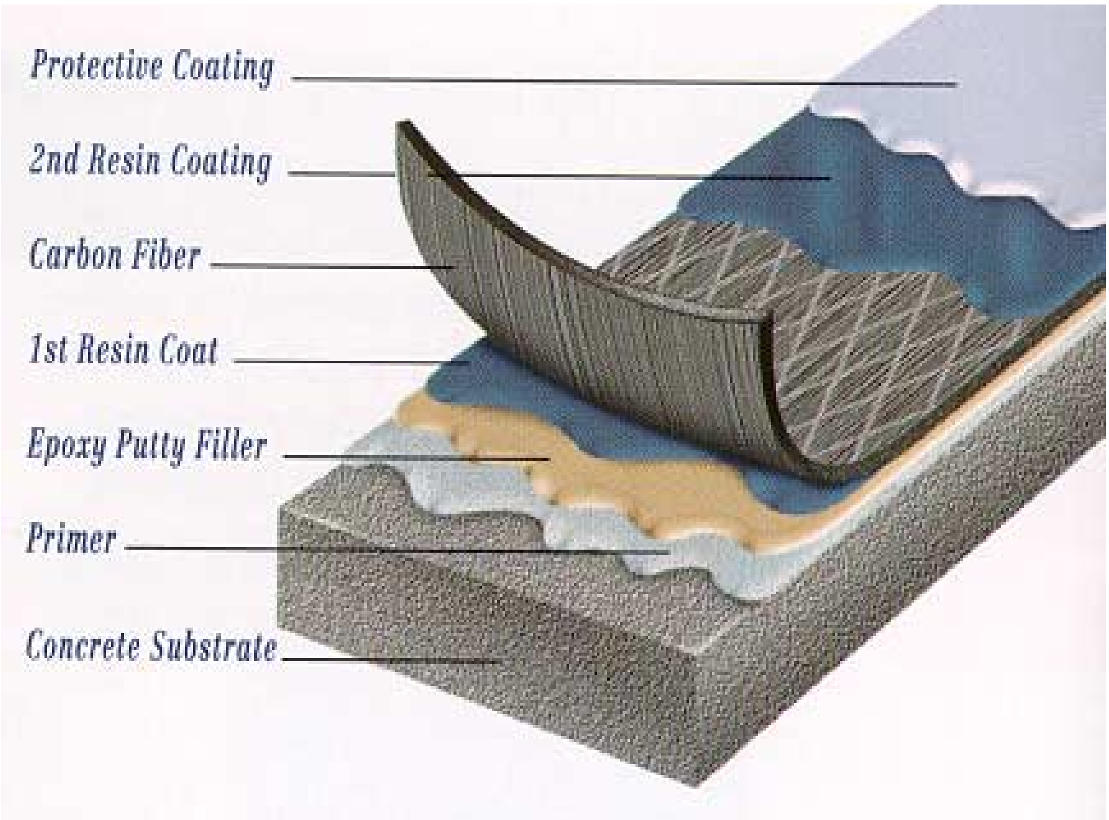


Figure 3: FRP - Installation Detail

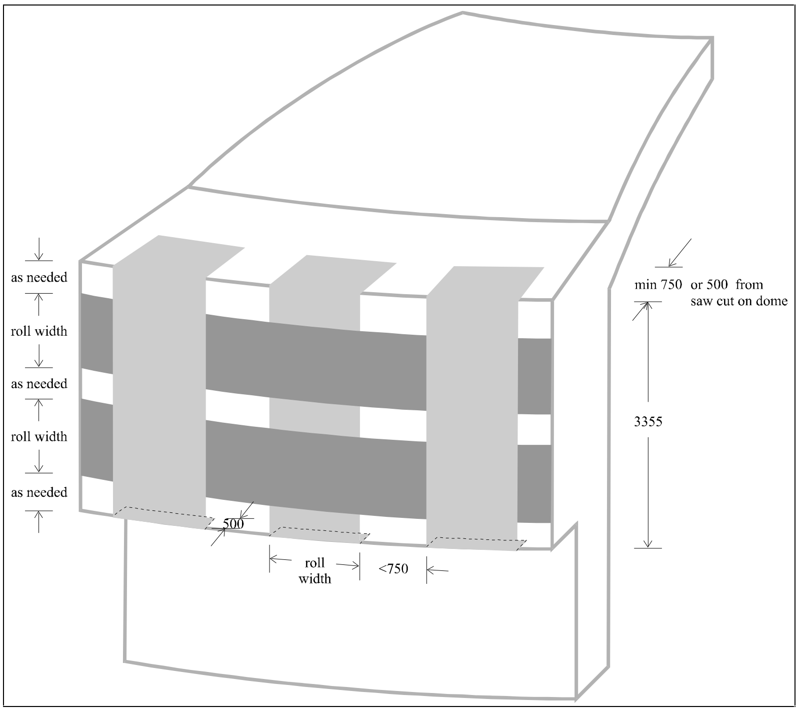


Figure 4: Ring beam FRP Installation Pattern

PROJECT EXECUTION

Concrete Demolition

As shown in Figure 5, using the sounding method, all of the unsound and unbonded concrete was marked, measured and prepared for demolition.

Figure 5: Sounding the Concrete Surface

The unsound concrete was removed using saw cuts and jack hammers, as presented in Figure 6. For shallow demolished sections (less than 50 mm deep), the saw cut around the perimeter of repair was at least 12 mm. For deep demolished sections (more than 50 mm deep), the saw cut was at least 25 mm. The maximum demolition depth into the ring beam was 0.66 m.

Figure 6: Concrete Demolition

After the demolition, and prior to the grouting/concreting, skin reinforcement was installed, Figure 7. Surface preparation by hydro-jetting or sand blasting was employed to open the pore structure of the concrete surface and to remove dirt and other debris material, Figure 8.

Figure 7: Installation of Skin Reinforcement

Figure 8: Surface Preparation –Hydro-Jetting

Concrete Repair - Materials and Application Methods

The concrete repair materials and repair techniques differ for shallow repairs and deep repairs. The shallow repair patches have an average thickness of less than 50 mm. The deep repair patches have an average thickness of greater than 50 mm.

The shallow patches are repaired with Sika Repair 225. The repair material is a prepackaged ready to use, cementitious, high strength, shrinkage compensated mortar, which includes silica fume and fibre reinforcement. The mix is used with the amount of water specified in the technical data sheet. The material was applied by hand troweling.

The deep patches are repaired with Sika Grout 212, SikaCem 810, PeaGravel (5-9 mm) and water. The patches are formed, the concrete mix is poured and vibrated while being placed in the forms as shown in Figures 9 and 10.

Figure 9: Deep Repair - Concrete Pour

Figure 10: Deep Repair – Completed

Glass Fiber Reinforcing Polymer (GFRP) Installation

For this application, the MBrace (Master Builder Technologies) Composite Strengthening System was chosen. Its function is primarily to protect and ensure durability of the concrete repairs. The structural strength and high modulus of elasticity were not critical requirements. Based on the material properties, E-glass fiber (EG-900) fabric was the selected material since it exhibited the best flexibility and elasticity of the system for this application.

The steps for the GFRP application are as follows:

Step 1: MBrace Primer (low viscosity to penetrate the concrete pore structure).
Step 2: MBrace Putty (high viscosity epoxy paste used for surface leveling).
Step 3: The 1st Resin Coating, MBrace Saturant (low sag epoxy that encapsulates the fibers).
Step 4: MBrace GFRP EG 900 E-Glass Fiber Fabric (instead of C-Fiber shown in Figure 3).
Step 5: The 2nd Resin Coating, MBrace Saturant (low sag epoxy that encapsulates the fibers).
Step 6: Protective Coating: Sonocoat Topcoat Super Colorcoat VOC Top Coat was used.

Site application of the GFRP to the ring beam is shown in Figure 11.

Figure 11: GFRP Installation

Quality Control

Quality control of the freshly mixed repair material included: water/cement ratio, slump of the mix, sampling the mix for the compression tests, compression test results. The 28 day compression test results ranged from f_c'=39 to 51 MPa, which exceeded the specified requirement of f_c'=30 MPa.

The quality workmanship and the concrete properties of the placed repairs was verified by drilling 101.6 mm diameter cores 'in situ' and performing pull-out tests as shown in Figure 12. The pullout tests gave average bond strength (repair material to old concrete) from 1.2 MPa to 2.1 MPa, which is above the revised value of 1.0 MPa required by the Specifications.

Figure 12: Pull-out Test

The quality of the installation of the GFRP and compliance with the Specification was also tested by performing pull-out tests on four 101.6 mm diameter cores. The pull-out tests gave average bond strength of the GFRP to substrata (repair material or original concrete) from 1.9 MPa to 3.4 MPa. That was also above the value required by the Specification. The failure mode was never through the GFRP/substrata contact joint, but rather through the original concrete, grout material or the repair material.

Figure 13: Completed Repair South-West View

Conclusions

Based on the results of the assessment studies and the repair work performed, it is concluded that:

1) The repair design and implementation of the Gentilly-1 ring beam has been successfully completed.
2) The repaired structure is in good condition and is satisfying current functional, safety, design and aesthetic requirements. The life of the structure has been extended for at least the next fifty years.
3) The structure should be closely monitored. The behavior of the repair materials will be compared with the original design intent, and the required maintenance effort will be performed to ensure that the design requirements characterized by service life are met.

Acknowledgements

The authors would like to thank Vector Construction Group for completing the Project on time and satisfying high quality requirements, and ISIS Canada for their technical support and expert opinion with the development of the technical specification.

The Repair of Nuclear Power Plant Reinforced Concrete Marine Structures and Installation of an Automated Cathodic Protection System.

LM Smith British Energy Generation (UK) Ltd
CA Hughes British Energy Generation (UK) Ltd
G Jones C-Probe Ltd

Abstract

This paper reports on a project which carried out repairs to the headworks and associated jetty and marine structures at Hunterston B nuclear power station. Although alternative sources of cooling are available, the headworks and associated structures have important availability functions with regard to the provision of cooling water for the nuclear power plant. These marine structures are situated in an exposed coastal location with consequentially aggressive environmental conditions. The jetty and headworks were originally designed to codes that have now been superseded and in order to ensure that the repaired headworks structure would have sufficient future service life an automated impressed current cathodic protection system was installed at the time of the repair works.

As a structural material concrete is strong in compression and weak in tension. In order that concrete may be utilised in practical structures, reinforced concrete is provided with steel reinforcement that carries any tensile loads or stresses by composite action. It is therefore important that the steel reinforcement in reinforced concrete structures is maintained in good condition otherwise the capacity of the structures will be degraded and reduced. The major threat to concrete structures is corrosion of the steel reinforcement and this is particularly the case in marine structures due to chloride contamination. The extent of corrosion may be monitored by measurement of the corrosion potential of the steel reinforcement relative to a reference electrode and cathodic protection may be employed.

The remedial works carried out to the jetty at Hunterston B are a typical example of the use of this type of repair and monitoring . The Hunterston jetty is the only means of access to the station's Cooling Water intake headwork structure. The intake headwork is essential as it provides the sea water that is used as the cooling water for the steam condensers within the power station. The approach jetty and the headwork were constructed using conventional reinforced concrete in the late 1950s and the early 1970s respectively.

During routine structural inspections it was identified that the structures required remedial works primarily due to the high degree of chloride contamination as a direct result of the environmental exposure conditions found at the site.

To ensure structural integrity over the remaining life span of the structures, an impressed current Cathodic Protection (CP) system was selected for the headwork structure above Mean Low Water Springs (MLWS) level and a sacrificial anode system below this level. The impressed current CP (ICCP) system on the structure above MLWS is divided into a number of anode zones and each zone is independently powered and monitored. The anode for all the zones comprises a mixed metal oxide coated titanium mesh fixed directly to the repaired and prepared concrete substrate with proprietary fixing pins which hold the mesh rigidly against the concrete.

This paper describes the repair works carried out to the headworks and the function, technical details, installation and performance of the cathodic protection system.

The Repair of Nuclear Power Plant Reinforced Concrete Marine Structures and Installation of an Automated Cathodic Protection System.

Introduction

This paper reports on a project which carried out repairs to the headworks and associated jetty and marine structures at Hunterston B nuclear power station. Although alternative sources of cooling are available, the headworks and associated structures have important availability functions with regard to the provision of cooling water for the nuclear power plant. These marine structures are situated in an exposed coastal location with consequentially aggressive environmental conditions. The jetty and headworks were originally designed to codes that have now been superseded and in order to ensure that the repaired headworks structure would have sufficient future service life an automated impressed current cathodic protection system was installed at the time of the repair works. This paper describes the repair works carried out to the headworks and the function, technical details, installation and performance of the cathodic protection system.

History

The remedial works carried out to the jetty at Hunterston B are a typical example of the use of this type of repair and monitoring. The Hunterston jetty is the only means of access to the station's Cooling Water intake headwork structure. The intake headwork is essential as it provides the sea water that is used as the cooling water for the steam condensers within the power station. The approach jetty and the headwork were constructed using conventional reinforced concrete in the late 1950s and the early 1970s respectively.

The water intake head works structure is of reinforced concrete construction as shown in Figures 1 and 2. During the mid-1980s repairs were carried out to the water intake headworks to remedy deterioration related to chloride ingress and reinforcement corrosion. These repairs were largely patch repairs but also included:

- Application of sprayed Gunite with an embedded secondary steel reinforcement mesh typically 125mm in depth to the entire soffit of the upper deck slab.

- Application of black pitch extended epoxy resin to the upper deck slab soffit.

- Application of an epoxy paint system to the columns and intake shaft above high water level only.

No cathodic protection was included in the repair scheme.

By the mid-1990s the repaired areas and some additional areas of the structure had begun to deteriorate once more and, during routine structural inspections, it was identified that the structures again required remedial works. This was primarily due to the high degree of chloride contamination as a direct result of the environmental exposure conditions found at the site which was aggravated by the dynamic effect of wave impact and storm damage on the .existing repairs

Repairs

As a structural material concrete is strong in compression and weak in tension. In order that concrete may be utilised in practical structures, reinforced concrete is provided with steel reinforcement that carries any tensile loads or stresses by composite action. It is therefore important that the steel reinforcement in reinforced concrete structures is maintained in good condition otherwise the capacity of the structures will be degraded and reduced. The major threat to concrete structures is corrosion of the steel reinforcement and this is particularly the case in marine structures due to chloride contamination. The extent of corrosion may be monitored by measurement of the corrosion potential of the steel reinforcement relative to a reference electrode and cathodic protection may be employed.

To ensure structural integrity over the remaining life span of the structures, an impressed current Cathodic Protection (CP) system was selected for the headwork structure above Mean Low Water Springs (MLWS) level and a sacrificial anode system below this level. The combined system therefore comprises:

- Sacrificial anode system to all exposed surfaces between +0.60m OD and –1.325m OD, and

- Titanium mesh impressed current anode and cementitious overlay to all surfaces above +0.60m OD

The impressed current CP (ICCP) system on the structure above MLWS is divided into a number of anode zones and each zone is independently powered and monitored. The anode for all the zones comprises a mixed metal oxide coated titanium mesh fixed directly to the repaired and prepared concrete substrate with proprietary fixing pins which hold the mesh rigidly against the concrete.

The structure was divided into six Remedial Work Areas (RWAs). A 40mm gap was left between the installed anode meshes in adjacent RWAs except in areas with double layer anode meshes where 50 mm of the top layer mesh was removed. All metallic objects within the protected areas were made electrically continuous with the steel reinforcement. A 25 mm sprayed concrete overlay was generally applied over the anode mesh, which was increased to 75 mm at the top deck. In some areas existing concrete was cut back to maintain both external clearances and the overlay thickness.

The impressed current cathodic protection system chosen was the C-Probe Achilles system (Figure 3) which allows the structure to be divided into a number of zones for both cathodic protection and corrosion monitoring by means of embedded electrodes. A zoned ICCP and monitoring system has superior performance to a single global system as the impressed current can be tailored to meet the requirements of separate areas of the structure.

Existing previous repairs to the deck slab soffit, including secondary steel mesh reinforcement, were removed and the surface was profiled to maintain a minimum of 30 mm cover to the main reinforcement, which was repaired as required. The sequence of removal and reinstatement operations (A through E) is shown in Figure 4. High pressure water jetting was used to remove the concrete and patch repairs (Plates 1 & 2). Where required, existing reinforcing bars that were excessively corroded were supplemented and/or replaced with new steel suitably anchored into the structure. Once the entire deck slab soffit had been reprofiled and prepared a titanium anode mesh and anode ribbons were fixed and a sprayed concrete overlay applied (Figures 5 & 6).

Performance

The latest repairs have performed very well without deterioration over a period of six years and have not shown the rapid deterioration that occurred with previous repairs. This is considered to be due to the inclusion of the zoned ICCP and monitoring system and it is recommended that future repairs to important structures in this type of location include such a system.

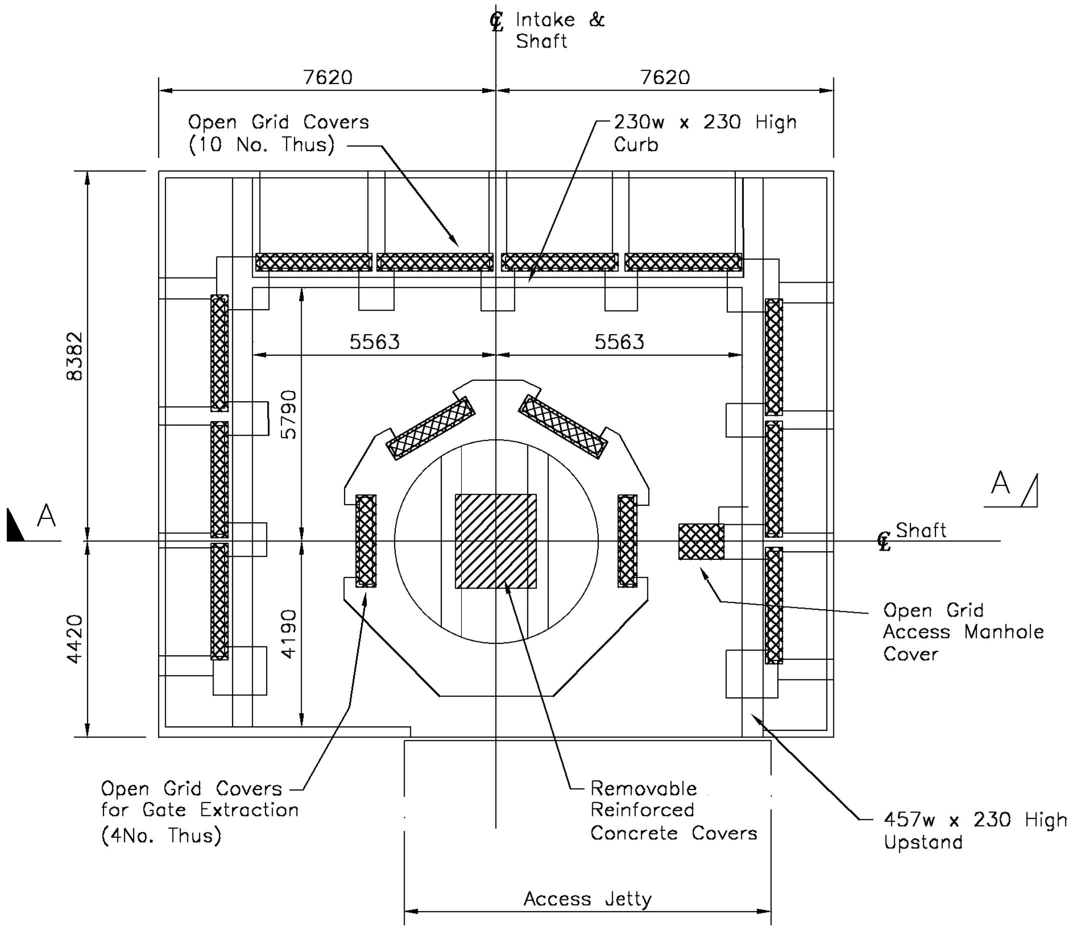


Figure 1

LAYOUT PLAN

(Scale 1:100)

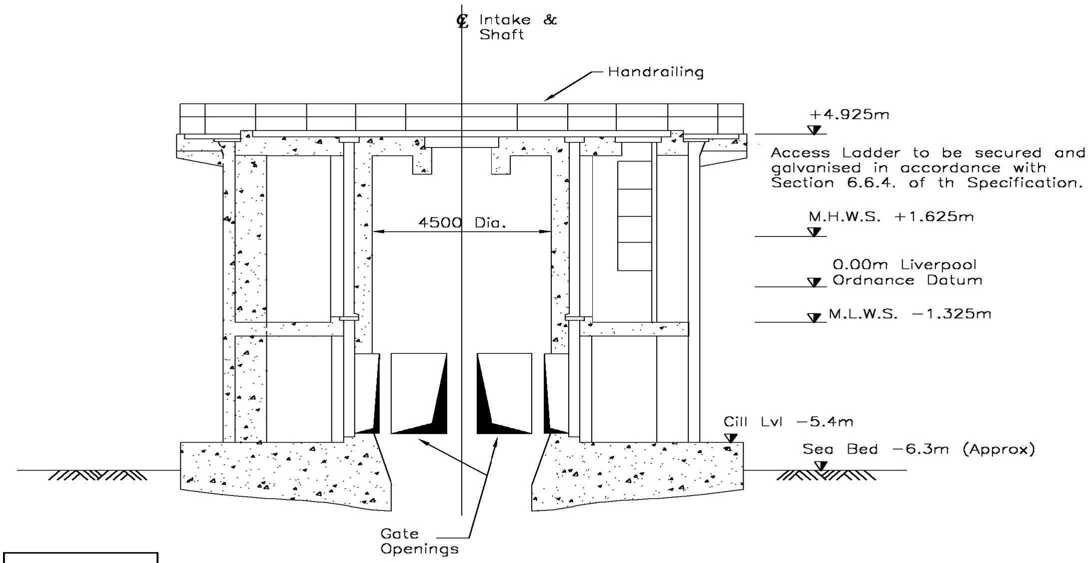


Figure 2

SECTION A – A

(Scale 1:100)

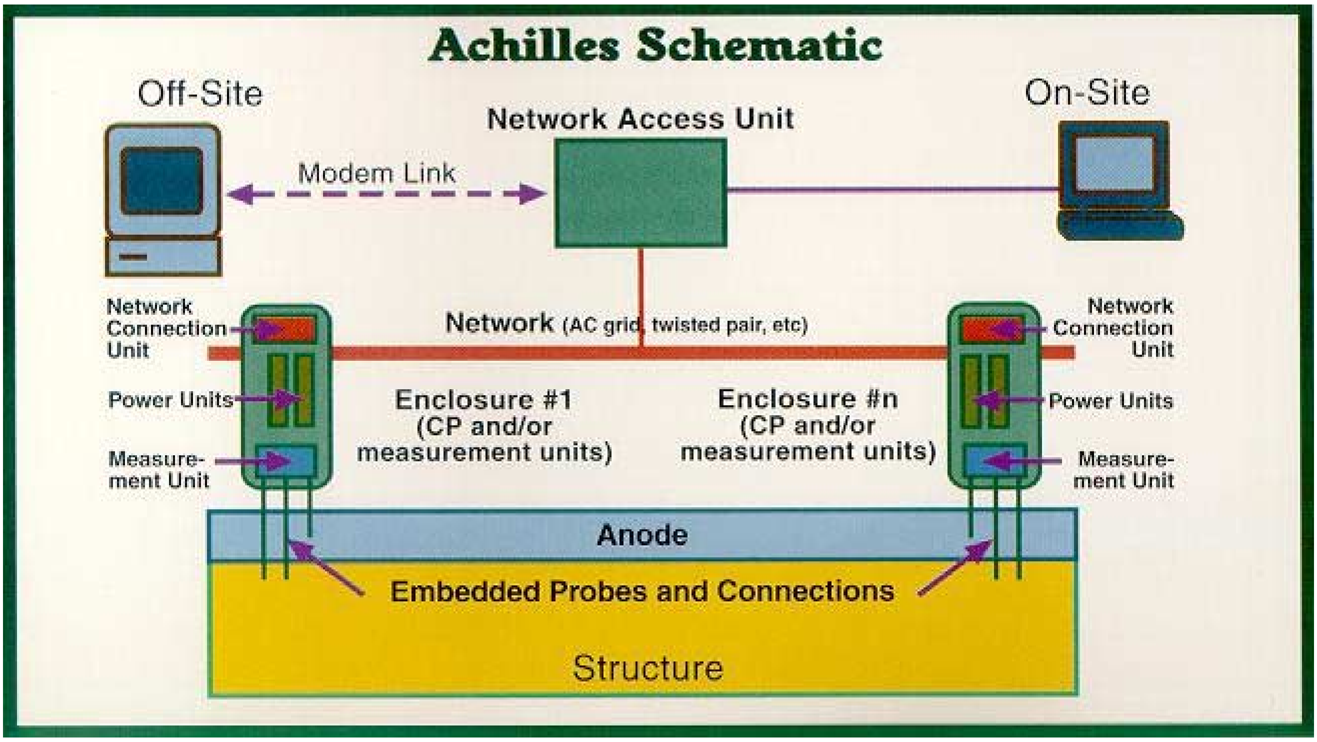
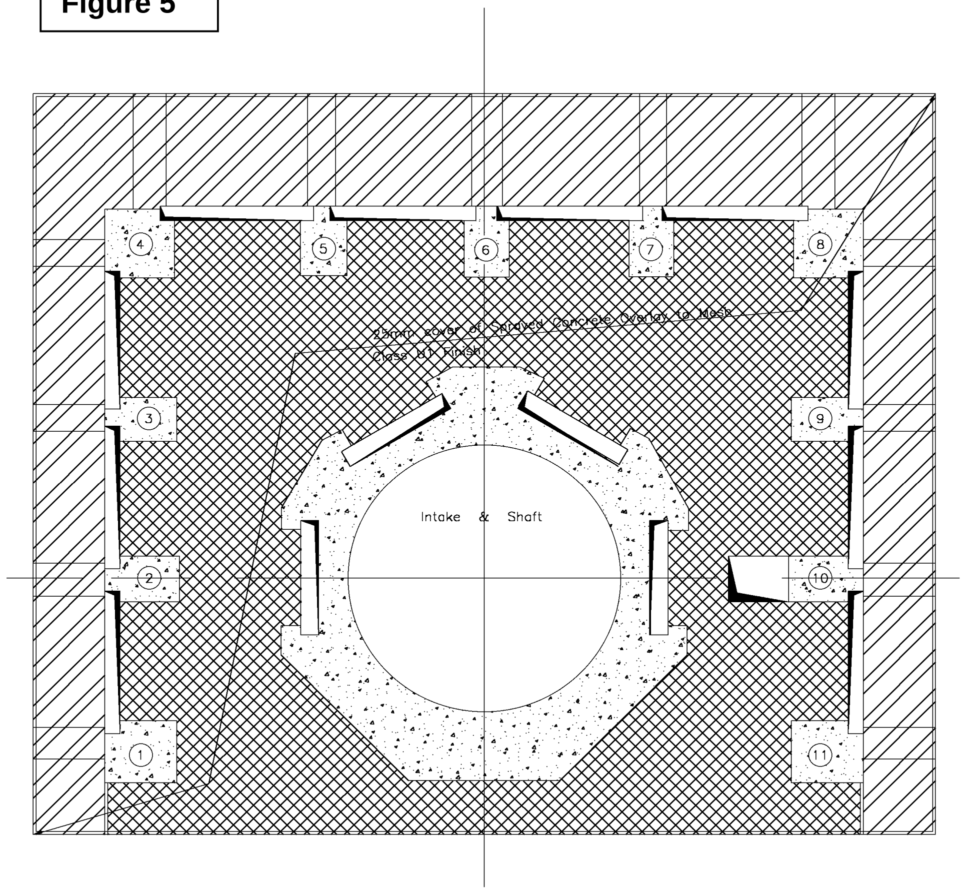

Achilles Schematic
Off-Site
On-Site
Network Access Unit
Modem Link
Network Connection Unit
Network (AC grid, twisted pair, etc)
Network Connection Unit
Power Units
Enclosure #1 (CP and/or measurement units)
Enclosure #n (CP and/or measurement units)
Power Units
Measure-ment Unit
Measure-ment Unit
Anode
Embedded Probes and Connections
Structure

Figure 3

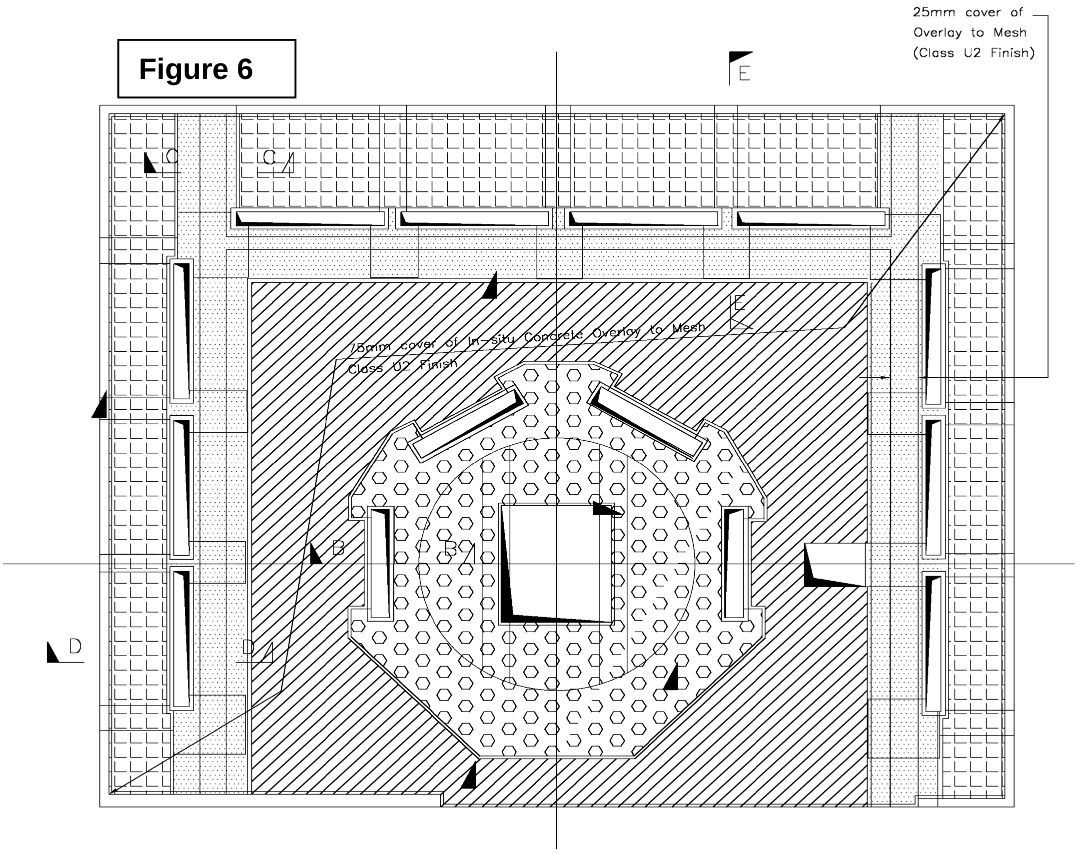
Figure 5



PLAN ON SOFFIT

(Scale 1:50)

Figure 6



PLAN ON TOP DECK

(Scale 1:50)

Plate 1   Water jetting

Plate 2 Exposed steel reinforcement after removal of cover

FEASIBILITY STUDY OF IE-SASW METHOD FOR THE NON-DESTRUCTIVE EVALUATION OF CONTAINMENT BUILDING OF NUCLEAR POWER PLANT

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Jong-Rim Lee, Korea Electric Power Research Institute, Korea
Jeong-Moon Seo, Korea Atomic Energy Research Institute, Korea

ABSTRACT

The IE-SASW method that combines Impact-Echo (IE) method with spectral analysis of surface waves (SASW) is proposed as a newly developed nondestructive testing method in concrete structures. This method is based upon the idea that IE method uses the elastic p-wave velocity measured from SASW method on the concrete member, and applied to specimens to evaluate its feasibility. It was shown that the thickness of the concrete structure member and the depth of the defects such as voids could be identified by IE-SASW method with good reliability. Additionally, the GPR (Ground Penetrating Radar) techniques have been applied to the same specimens in order to establish the performance and reliability of the proposed method, and compared with IE-SASW method. The experimental studies show that it is more preferable to use the IE-SASW than GPR to detect the voids just beneath the steel reinforcing bars that may exist in concrete structures.

Keywords: Non-destructive Testing, Concrete, Impact-Echo, SASW

1. INTRODUCTION

The construction quality of the containment building in the nuclear power plant is carefully controlled and thoroughly inspected to prevent from unexpected flaws. In general, the concrete of containment building is deteriorating as time passes by, so the periodic safety assessments using non-destructive tests are also crucially required. Until now, the non-destructive tests such as radar, impact-echo, and ultrasonic methods have been developed for the concrete structure and compared their characteristics by various test specimens (M. Krause. et al., 1997). Even though each non-destructive testing method has its own advantage and capability, it is necessary for the user to understand the limitations of the methods exactly before applying them. Also, the best combinations of testing methods can be selected only after comparative studies are performed.

In this study, IE-SASW testing method was introduced and the performance was evaluated by applying to the containment building of nuclear power plant. The IE-SASW method combines IE (impact-echo) with SASW (spectral analysis of surface waves) methods, and is applied to detect the flaws as well as the thickness of concrete structures. In the original IE method, the thickness and flaw depth in the concrete structure are determined using the predetermined P-wave speed (Sansalone, 1997). The accuracy of the result is much dependent on the accurate measurement of representative P-wave speed of the testing location. In the IE-SASW method, the representative P-wave speed is determined using SASW method.

In this paper, the performance of applying IE-SASW method to the non-destructive evaluation of containment building of nuclear power plant was studied. Two test specimens were constructed and the defects were included at the known locations. One of the specimens was the prototype of a structural member of the containment building typically built in Korea. IE-SASW method was performed to locate the flaws and to determine the thickness.

Additionally, the GPR (Ground Penetrating Radar) method is applied to the same specimens, and compared with IE-SASW method. The series of experimental studies were focused to detect the void just beneath the steel reinforcing bars that may happen in concrete structures.

2. NON-DESTRUCTIVE TESTING METHODS USING ELASTIC WAVE

2.1. Impact-Echo Method

Impact-echo method is a nondestructive testing method of concrete structures that is based on the propagation characteristics of impact-generated stress waves that are reflected by internal flaws and external surfaces. It can be used to determine the location and extent of flaws such as cracks, delaminations, voids, honeycombing, and debonding in reinforced concrete structures (Sansalone, 1997). A schematic diagram of impact-echo method is shown in Figure 1.

The stress pulse generated by an impact on the surface propagates back and forth between the internal interface and top surface of a test object. Surface displacements caused by reflections of these waves are recorded by transducer (accelerometer) located adjacent to the impact. The resulting displacements versus time signals are transformed into the frequency domain, and plots of amplitude versus frequency spectra are obtained. Multiple reflections of stress waves between the impact surface, flaws, and/or other external surfaces give rise to transient resonance, which can be identified in the spectrum, and used to evaluate the integrity of the structure or to determine the location of flaws. The typical wave forms and amplitude spectrum of signal are illustrated in figure 2.

The period of reflected waves is equal to the travel path 2T, divided by the compressive wave velocity, Vp. Since the frequency is the inverse of the period, the resonance frequency f, is:

$$f = V_p / 2T \tag{1}$$

This is a fundamental equation of impact-echo response for solid member, and if the Vp is pre-determined, the thickness and location of internal flaws can be identified.

The above analysis is valid for the cases where the reflecting boundary or internal interface has lower acoustic impedance (density × P-wave speed) than the member. When this case occurs, P-wave incident upon the interface changes sign. For example, the P-wave generated by impact is a compression wave. When this wave is incident upon a solid/air interface, the reflected wave is a tension wave as illustrated in figure 3(a). If, however, a P-wave is incident upon an interface that has higher acoustic impedance, such as steel reinforcing rebar in the concrete, or enlarged area, the incident P-wave does not change sign, and tension wave will be reflected at the interface (higher acoustic impedance) as a tension wave as illustrated in figure 3(b). Thus equation (1) must be modified, because the period is twice as long.

$$f = V_p / 4T \tag{2}$$

2.2. Spectral Analysis Of Surface Waves (SASW) Method

The spectral analysis of surface waves (SASW) method is a method of seismic testing that has developed for determining shear wave velocity profiles at soil and/or pavement sites (Nazarian and Stokoe ii, 1986). The SASW method is a nondestructive method in which both the source and receivers are located on the surface as shown in figure 4.

The source is simply a transient vertical impact that generates a group of surface waves of various frequencies in the medium. Two vertical receivers located on the surface monitor the propagation of surface wave energy. By analyzing the phase information of the cross power spectrum determined between the two receivers, surface wave velocity – wavelength relation is determined. If the stiffness of a site varies with depth, then the surface wave velocity will vary with wavelength. The variation of surface wave velocity with frequency (wavelength) is called dispersion, and a plot of surface wave velocity versus wavelength is called a dispersion curve. The dispersion curve is developed from phase information of the cross power spectrum. This information provides the relative phase between two signals (two-channel recorder) at each frequency in the range of frequencies excited in the SASW test. In a homogeneous medium, surface wave velocity, V_r, is constant and independent of the wavelength. Detailed procedure of determining dispersion curve in the SASW method is described elsewhere (Joh, 1996, Nazarian and Stokoe ii, 1986).

In order to apply the SASW technique to the nondestructive test in concrete structures, the variation in surface wave velocity along the whole thickness of a concrete structure should be determined, and the P-wave velocity can be converted using Equation (3), assuming the Poisson's ratio of the concrete to be in the range between 0.15 and 0.2.

$$V_p = \frac{1+\nu}{0.87+1.12\nu}\sqrt{\frac{2(1-\nu)}{(1-2\nu)}}V_R \qquad (3)$$

The probable error in determining P-wave velocity with the assumed Poisson's ratio of concrete between zero and 0.2 is about 3%, which is minimal, and with Poisson's ratio of 0.2, the Equation (3) reduces to:

$$V_p = 1.79V_R \qquad (4)$$

2.3. IE-SASW method

In order to predict the thickness of concrete member or to identify the defect using IE method, P-wave velocity of concrete is required. The P-wave velocity can be found as presented in Equations (1) and (2), when the boundary condition and the thickness of concrete member are predetermined. In general, however, the thickness of member such as slab and wall of building remains unknown. The concrete specimen should be extracted from the structures using core-boring machine, and substituting the height of core specimen into the equations (1) produces the P-wave velocity. But this method has such disadvantages that the surface of structure is destructed due to core boring, and the P-wave velocity calculated may not be representative value of the structures because of the non-homogeneity of the concrete material.

Therefore, IE-SASW method that enables to obtain the P-wave velocity from SASW method nondestructively and then to apply the IE method, is proposed in this study. In order to obtain the material properties in the multi-stratified soil system using SASW method, the iterative inversion processing is required until the discrepancy between the experimental dispersion curve and theoretical dispersion curve is minimal (Joh, 1996). But in concrete structures, the surface wave velocity (V_r) can be easily obtained by experimental dispersion curve without executing the inversion procedure, because concrete material is assumed to be composed of a single layer. Using the relationships among surface wave velocity (V_r), P-wave velocity (V_p), and Poisson's ratio (ν) with assumed Poisson's ratio of 0.20, P-wave velocity of concrete can be determined by Equation (4). Then, the thickness or the location of defect such as void in the concrete member can be identified using IE method nondestructively.

3. APPLICATIONS

3.1 Test Specimens

Two test specimens were made to study the feasibilities of IE-SASW method in the nondestructive evaluation of reinforced concrete structures. One test specimen, named “A” (length: 150cm, width: 50cm, thickness: 30cm, including two voids) was designed as illustrated in figure 5. Voids were simulated with styrofoam of which the acoustic impedance is distinguishably smaller than that of concrete. Especially, in this specimen, one void is located just beneath the rebars and the other is located apart from the rebar. The cover depth to the void beneath the steel reinforcing bar is 10cm and the other is 15cm. The mix proportion of the concrete is shown in table 1.

In order to obtain the P-wave velocity, three concrete test molds were made and cured in a water bath at 20°c for 3 days. After casting, the P-wave velocity was measured using IE method respectively, and averaged to provide the P-wave of about 3300 m/sec.

Another test specimen, named “B”, is prototype structural member of the containment building of a nuclear power plant with three tendon sheathing pipes (diameter of 150mm) that are unfilled and some dozens of rebars (diameter of 55mm) as shown in figure 6. In order to compare the feasibilities of both IE-SASW and GPR methods, the cubic styrofoam (10cm $\times$ 10cm $\times$ 10cm) and water container (width: 20cm, height: 40cm, thickness: 40cm) filled with water, and a PVC pipe which is 15cm in diameter were inserted intentionally during the construction at the depth of 30cm from the surface of the specimen. The photograph 1 shows the prototype member of the containment building of a nuclear power plant.

3.2 Results of IE-SASW method

3.2.1. Test specimen-A

The SASW test was performed on the test specimen-a to get the p-wave velocity before applying the IE test. The interval between source and the first receiver is set to 20cm that is the same distance between receivers. Figure 7 shows the dispersion curve produced by the phase difference between the two receivers, and the phase velocity is approximately chosen out to be 1870m/sec. The phase velocity between two receivers is equal to surface wave velocity without inversion process, because it can be assumed that the surface wave velocity is constant and independent of wavelength in a concrete layer. Thus, the P-wave velocity can be calculated as 3347m/sec by the equation (4), assuming the Poisson’s ratio of concrete to be 0.2. It is resulted that the P-wave velocity of 3,300 m/sec obtained from the SASW test shows good agreement with the velocities that are previously obtained from the concrete test molds and from IE test using known thickness. Therefore, it is revealed that the p-wave velocity can be obtained from the SASW test reliably.

The IE test was performed at the three positions: (1) on the void caged behind steel reinforcing bars, (2) on the void where there is no rebar, and (3) on the surface where there is no defect through test specimen. Several steel balls were used as impact sources, and accelerometer (PCB 353b15) was used as a receiver. Signals were recorded and analyzed using a dynamic signal analyzer (HP 35665a). The amplitude spectrums of acceleration at these three positions are presented in figures 8 (a), (b), and (c), respectively.

The location of the void and the thickness of specimen can be determined by amplitude spectrums using P-wave velocity determined by SASW method, and substituting these resonance frequencies in figures 8 (a), (b), and (c) into equation (1) produces 10.6cm, 15.1cm, and 30.1cm, respectively. Comparing these results with true values of 10.0cm, 15.0cm, and 30.0cm, it can be concluded that the IE-SASW method shows a good potential to identify the defect and unknown thickness of concrete member reliably.

3.2.2. Test Specimen-B

The P-wave velocity of test specimen was determined by the SASW test. A test was carried out along the line where there were no defects and no rebars as designated in figure 6. SASW test was performed twice, changing the interval between source and the first receiver from 20cm to 40cm. The dispersion curves are presented in figures 9 (a) and (b). The P-wave velocities calculated from the surface wave velocities were 4200m/sec and 4100m/sec, respectively, and average value of 4150m/sec was used in IE test.

A series of IE test were performed at the position (E1 to E13) designated in figure 6, and the amplitude spectrums of acceleration are shown in figure 10. At position of E1, the resonance peak frequency of 1824hz was observed, and the thickness of wall was calculated as 1.14m, which is in good agreement with the true thickness of 1.2m. At the position of E5, E6, E11, and E12 where the metal sheath pipes are located, the clear and large resonance peak frequencies were observed in the range from 4700hz to 4900hz. These frequencies produce the depth to the metal sheath pipe as 0.45~0.47 m, which fits well with the actual depth of 0.45m, and it is revealed that the metal sheath pipe have lighter impedance than concrete and acts as a free boundary. However, the impact-echo response from the rebar of position E2 shows the dominant peak at 4700hz, which does not give the actual depth to the steel reinforcing bar. The acoustic impedance of rebar is about five times that of concrete, but the area of rebar is much less than that of concrete. In this case, the reflected and refracted P-waves from the interfaces of concrete/steel or steel/concrete created by the rebar makes the signal to be complicated, because the location of rebar is shallow but the depth to the bar is relatively deep. This fact makes it difficult to locate the rebar by IE test.

the amplitude spectrums corresponding to styrofoam, water container, and PVC pipe are shown in figures 11 respectively. The impact-echo responses from these objects show large amplitude resonance between 6khz and 7khz. At the position of p1 in PVC pipe, the highest amplitude peak occurs at 6.976khz: this peak is used to calculate a depth of 0.297m, in good agreement with the known depth of 0.3m. At the position of s4 in styrofoam, the highest amplitude peak occurs at 6.336khz: this peak is used to calculate a depth of 0.327m, which is close to the known depth of 0.3m. Finally, at the position of w4 in water container, the highest amplitude peak occurs at 6016khz: this peak is used to calculate a depth of 0.345m, which is a little larger than the actual depth of 0.3m. These results show that it is feasible to use the IE-SASW method to detect the voids (filled with either water or not) and PVc pipe.

2.4. Results of GPR method

2.4.1. Test specimen-A

A ground penetrating radar (GPR) was employed and tested on both specimens A and B for the comparison purpose. Figure 12 shows the typical processed image corresponding to the profile of test specimen-a, which recorded with 1200mhz antennas. This section shows reflected and diffracted signals (hyperbolas designated by arrows) in response to the different objects: (1) six rebars, and (2) the void where there is no rebar. Contrary to the ie method the void beneath reinforcing steel bars could not be detected by GPR method.

2.4.2. Test specimen-B

The GPR profile corresponding to the test specimen-b is shown in figure 13. A series of rebar spacing evenly is illustrated as hyperbolas. However, it is difficult to identify the tendon sheath located behind the rebar, because most of the electromagnetic wave is reflected when encountered with rebar. Pulse type dipole antenna of 1,200mhz (central frequency) failed to detect the objects such as PVC pipe, styrofoam, and water container in the specimen-b.

3. Conclusions

The comparative studies presented here illustrate the performance and feasibility of nondestructive test to the concrete structures. The experiments were quite useful to select the most suitable techniques for specific applications. In this study, the IE-SASW method and GPR method are employed to evaluate their feasibilities. The conclusions obtained are as follows:

(1) The IE-SASW method can be applied to identify the location of defect and the thickness of concrete structure with good accuracy.
(2) Especially, the void just beneath the rebar in test specimen-a could be easily detected by the IE-SASW method. On the other hand, the rebar itself could not identified by this method.
(3) The location of tendon sheathing and thickness of the structural member (test specimen-B) could be identified by the IE-SASW method. Also the location of the objects such as styrofoam, water container, and PVC pipe that were intentionally included in the test specimen-B can be detected by this method.
(4) The GPR method provides an objective and reliable image corresponding to the rebars and defects such as voids. But it was difficult to identify the void just beneath the reinforced steel bars. Therefore, the detection potential can be improved by the combined utilization of the IE-SASW method and the GPR method.

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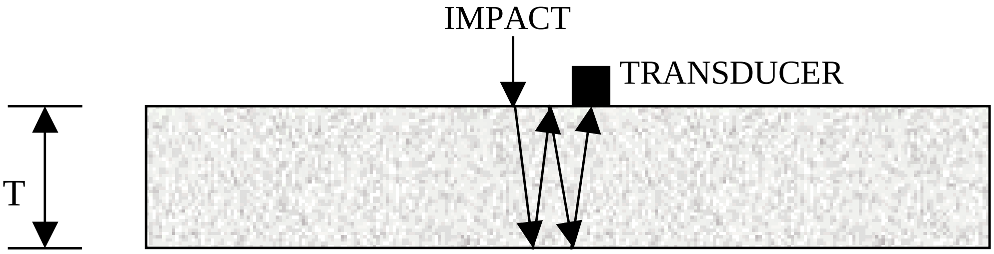
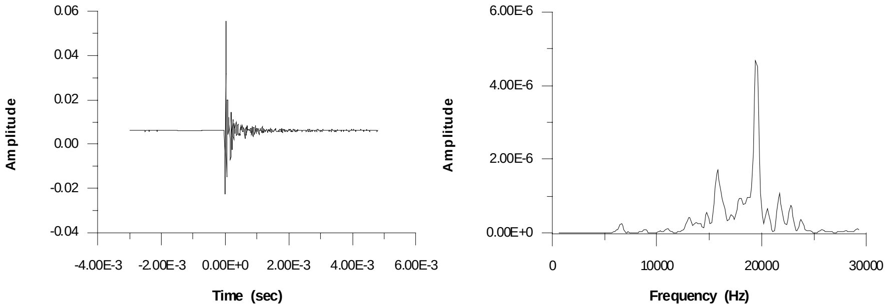
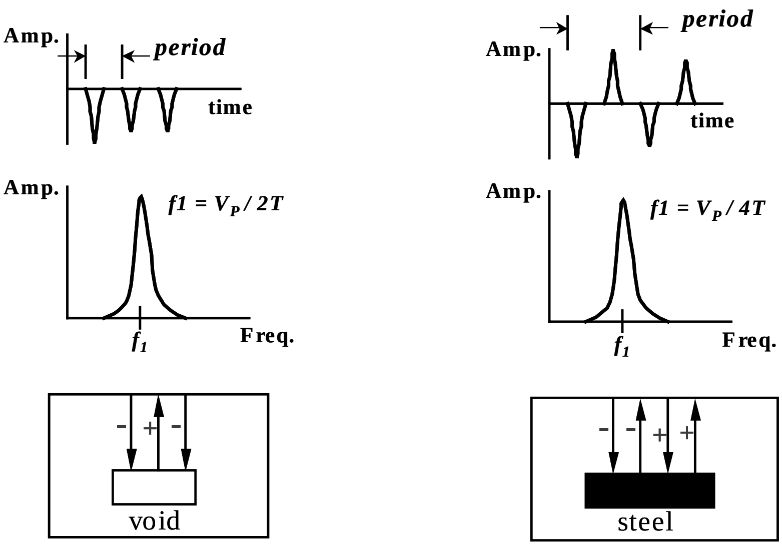


Figure 1. Simplified diagram of impact-echo method



(a) waveform (b) amplitude spectrum

Figure 2. Typical result of impact-echo test



(a) lower acoustic impedance (b) higher acoustic impedance

Figure 3. Comparisons of resonance frequency between lower and higher acoustic impedance

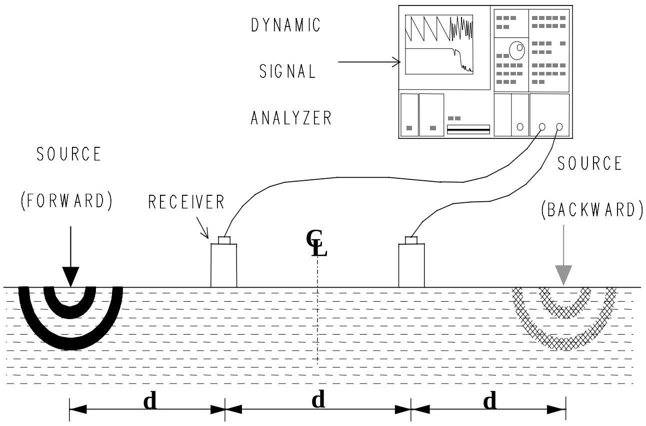


Figure 4. General configuration of SASW testing

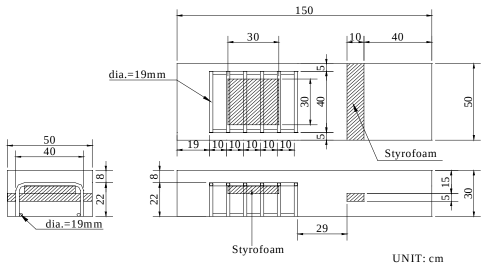


Figure 5. Drawing of test specimen-A

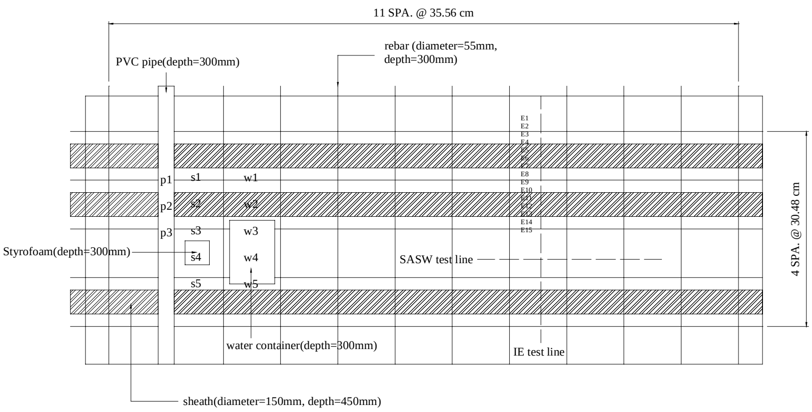


Figure 6. Drawing of test specimen-B that is the prototype structural member of containment building of nuclear power plant

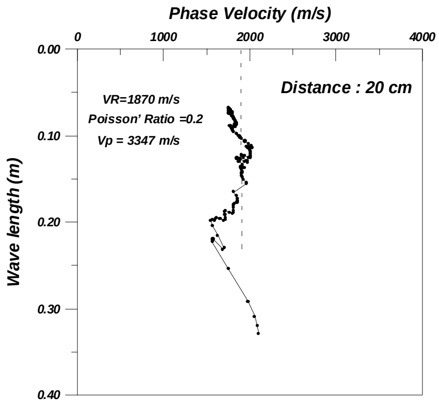
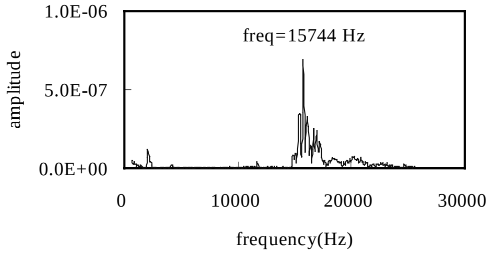
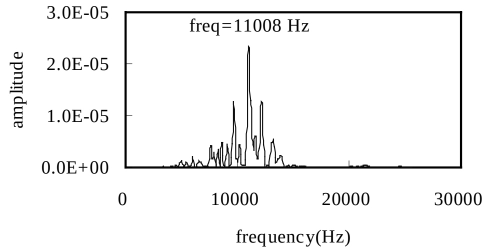


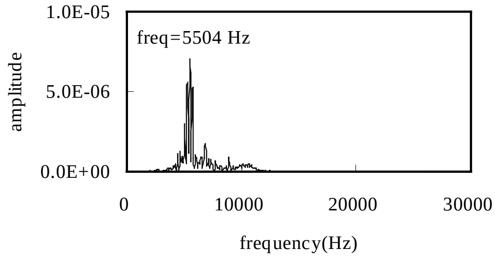
Figure 7. SASW test result to determine the P-wave velocity of test specimen-A



(a) amplitude spectrum for the void behind steel reinforcing bars



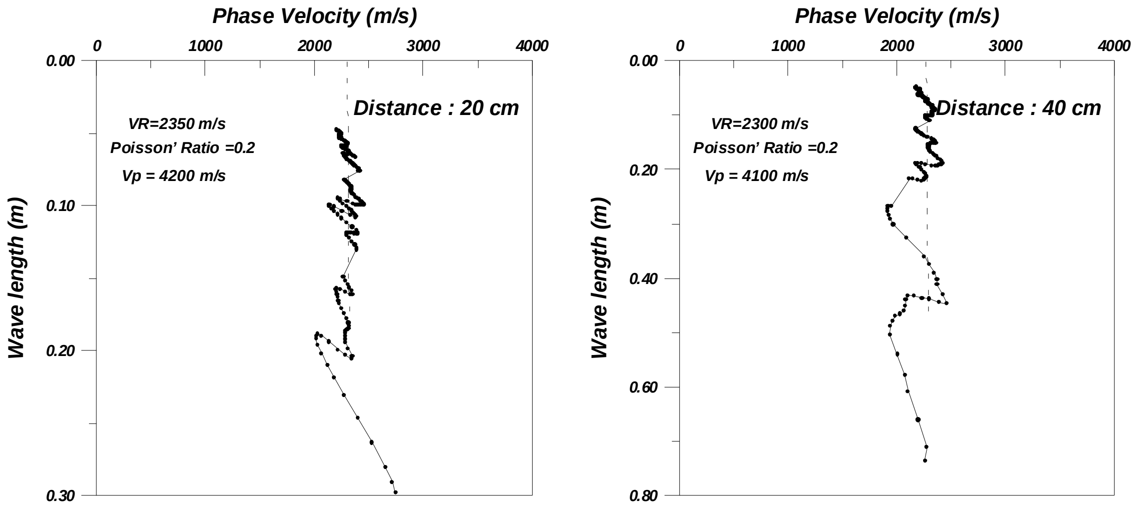
(b) amplitude spectrum for the void where there is no steel reinforcing bar



(a) on the surface where there is no defect through test specimen

Figure 8. Amplitude spectrums of acceleration for test specimen-A at the three positions:

(a) on the void behind steel reinforcing bars, (b) on the void where there is no steel reinforcing bar, and (c) on the surface where there is no defect through test specimen.



(a) Distance between receivers: 20cm (b) Distance between receivers: 40cm

Figure 9. SASW test results to determine the P-wave velocity of test specimen-B

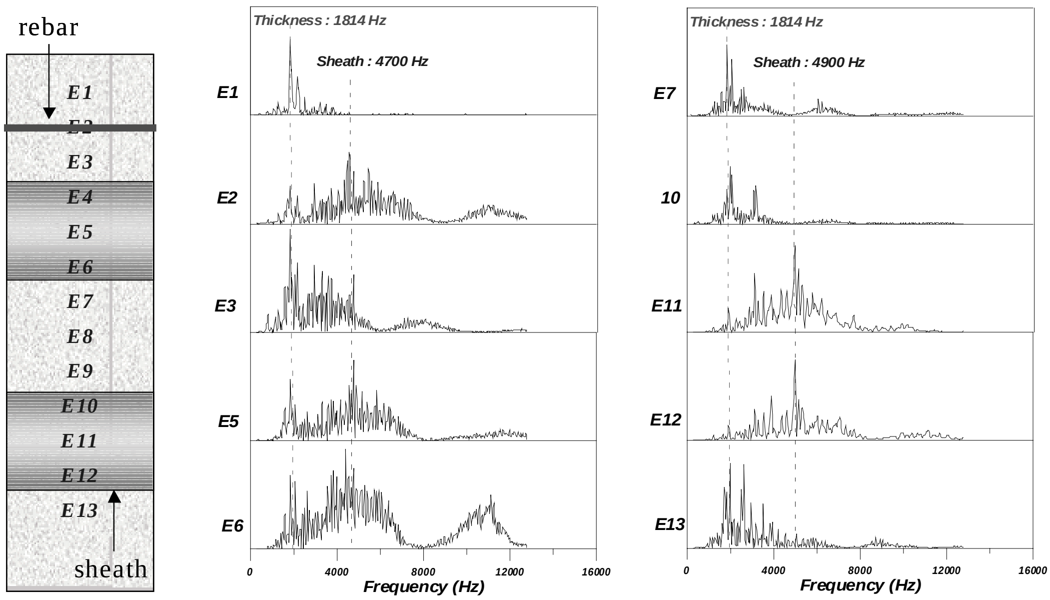
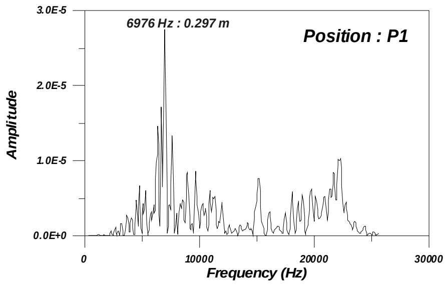
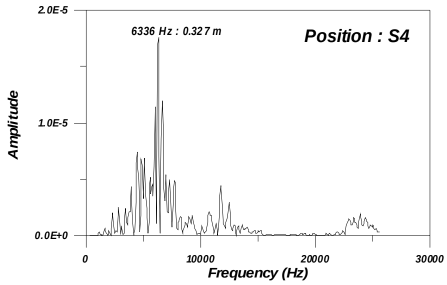
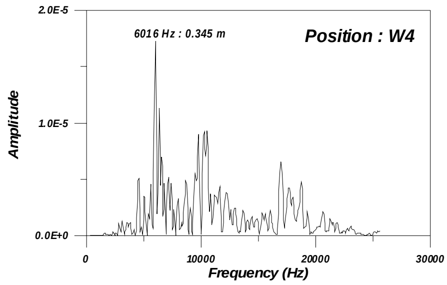


Figure 10. Amplitude spectrums of acceleration for test specimen-B at different positions (Theoretical resonance frequencies corresponding to wall thickness and sheath location were plotted as dotted lines respectively.)



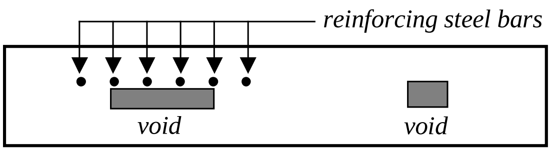


(a) Amplitude spectrum corresponding to PVC pipe (B) Amplitude spectrum corresponding to Styrofoam

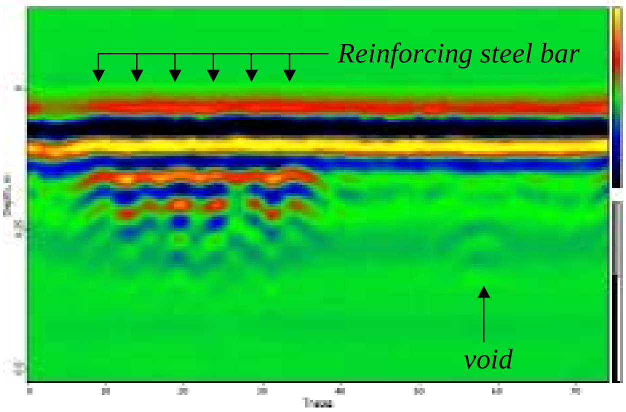


(c) Amplitude spectrum corresponding to water container

Figure 11. Amplitude spectrums of acceleration for PVC pipe, Styrofoam, and water container in test specimen-B at different positions

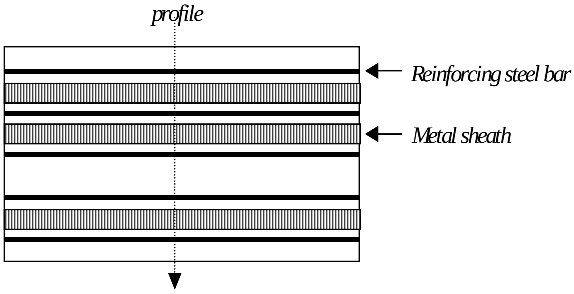


(a) Schematic plan showing the location of reinforcing bar and voids

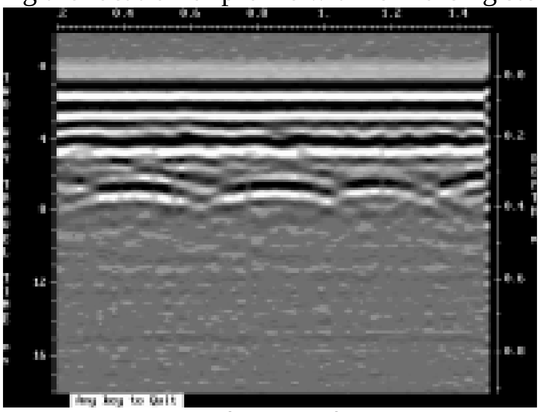


(b) Radar image of the reinforcing steel bars and void

Figure 12. GPR image at test specimen-A (scanned by 1200MHz antenna)



(a) Schematic plan showing the location of profile with reinforcing steel bars and metal sheaths



(b) Radar image of the reinforcing steel bars

Figure 13. GPR profile at test specimen-B (scanned by 1200MHz antenna)

Table 1. Mixture proportion of test specimen concrete

Water	Cement	Aggregate		S/A(%)	Air Content (%)	W/C (%)
		Coarse	Fine			
185	320	1026	713	41	5	58

Photograph 1. Prototype Structural Member of Containment Building Of Nuclear Power Plant

Field Studies of Effectiveness of Concrete Repairs

NJR Baldwin, Mott MacDonald, UK

Abstract

This paper presents a summary of the work carried out under an HSE research study entitled 'Field Studies of Effectiveness of Concrete Repairs'.

There is little published information describing or comparing the long-term performance of different repair types. This project examines 45 sites where repairs have been carried out and have remained in service for several years and evaluates the effectiveness of a range of concrete repair systems as applied in practice. The sites include a range of structure types, ages and service environments, and include bridges, tunnels, building frames, and car parks in industrial, public, highway and nuclear environments. The repairs include hand and trowel applied materials, sprayed materials, and cathodic protection techniques, and also some sites with coating and crack injection systems.

The objective is to improve practices for maintaining and improving the integrity of operational structures and so achieve higher standards of structural safety and reliability and better whole-life structural management.

The project employed a range of visual, non-destructive and destructive investigation techniques at repaired sites and compared the condition found with records of the repair procedures and objectives. The site investigations involved detailed visual inspection, and surveys using hammer tapping, covermeter, half-cell and carbonation depth, pull-off testing and core sampling for petrographic analysis of the repair material, repair layer interfaces and repair/substrate interface.

Examination of records of the repairs allowed assessments to be made of the level of understanding of the original cause of deterioration and the need and objectives for the subsequent repairs. Comparisons have been made between the specification and evidence from the repair sites. The owner's repair objectives and constraints, and the quality and effectiveness of the repairs have been considered.

The site investigations have provided detailed information on the performance, structure and effectiveness of repairs. Full records of the repairs were characteristically difficult to obtain. Most repairs were to corroded reinforcement in structures affected by chloride-induced corrosion. There was often evidence of post-repair corrosion with the exception of those structures with a cathodic protection system. Fine cracks and surface crazing were common in the repair materials and at the perimeter of patch repairs. These can penetrate to the reinforcement and represent performance limiting features.

The project output includes production of guidelines covering the decision making process of concrete repair. These guidelines will be disseminated to industry via the HSE and through new guidance notes within industry documents and a proposed ICE publication.

1. Introduction

1.1 Origin

Mott MacDonald Ltd (MM) was commissioned by the UK's Health and Safety Executive (HSE) in June 2000 to carry out a research study entitled 'Field Studies of Effectiveness of Concrete Repairs'. The project follows on from the project conducted by the UK's nuclear industry entitled 'Concrete repair materials and protective coating systems' that produced a compendium of repair materials and systems[1].

In 1999 the effectiveness of *in situ* repairs was added to the HSE's Nuclear Research Index. The issue questioned the potential performance, maintainability and longevity of nuclear safety related structures with extended service and safe storage lives using current concrete repair practices. HSE's Nuclear Industry Inspectorate (NII) proposed a programme of field studies to examine typical repair sites. NII recognised the difficulties in gaining access to repair sites within nuclear facilities and in 2000 secured funding from the HSE to conduct field studies on the repairs to a variety of concrete structures away from nuclear sites.

The scope and objectives have been developed between the HSE, MM and other organisations whose interests are represented in an Expert Group associated with the project. Funding has also been received from the Highways Agency (HA) and the Institution of Civil Engineers (ICE) Research and Development Enabling Fund. The project receives substantial additional support from collaborating organisation and individuals, as well as the co-operation of owners of repaired structures.

1.2 Objectives

The aim of this project is to evaluate the effectiveness of a range of concrete repair systems as applied in practice, in order to improve practices for maintaining and improving the integrity of operational structures and so achieve higher standards of structural safety and reliability and better whole-life structural management. This includes assessment of the whole process whereby repair is carried out, and in particular what parts of the process lead to success or failure. The project also investigates the effects of ageing of repairs and identifies the most effective means of providing enhanced durability. Guidelines covering the decision making process of concrete repair will be disseminated to industry.

The project has focussed primarily on non-structural patch repairs to reinforced concrete intended to arrest deterioration resulting from the ageing processes prior to any significant effect on structural integrity such that structural intervention is avoided. Structural repairs, in which the load paths pass through the repair, have not been specifically targeted.

Investigations have also been carried out into the long-term performance of practical cathodic protection (CP) schemes. The performance of the systems has been assessed, and the effects of CP on the bond of reinforcement and the distribution of ions throughout the concrete has been investigated.

1.3 Definitions

The terms used within the study have been the subject of intense debate. The problems lie in finding definitions which are agreeable for civil, structural and materials engineers, the repair industry and operators of structures, for terms such as defect, repair, effectiveness, performance, non-structural repair and structural repair. Examples of definitions from ENV 1504-9[2] and EU FP5 research project "LIFECON" are presented in Table 2.

Table 1 Definitions

	ENV 1504-9	EU FP5
Defect	An unacceptable condition which may be in-built or may be the result of deterioration or damage.	
Repair	A measure which corrects defects.	Return of a structure to an acceptable condition by the renewal, replacement or mending of worn, damaged or degraded parts.
Protection	A measure which prevents or reduces the development of defects.	

The definition and objective of repair works are also variable and the following definitions are found in the literature:

- Etebar[3] defines the objective of repair as being to restore or enhance one property such as durability, structural strength, function or appearance.

- Walker[4] states that rehabilitation involves controlling degradation to enable a structure to continue to serve its intended purpose, either through repair to a state similar to the original, or using methods to arrest deterioration processes.

- Emmons and Vaysburd[5], state that the object of any repair project is to *"produce a repair at relatively low cost with a limited and predictable degree of change over time and without deterioration and/or distress throughout its intended life and purpose"*.

These demonstrate the different elements and concepts of the objectives of repair and intended repair performance. Measurement of repair effectiveness is more complex and subjective, and is best addressed by comparing condition over time relative to the original objectives.

1.4 Effectiveness

A repair may be effective if it has achieved the performance that it was originally intended to. However, there may be several aspects to the original intention, such as cost, longevity and cosmetic issues. There may also have been requirements or restrictions for preparation and application of the repair, that form part of it's effectiveness. This project has used a detailed definition of effectiveness, adopting the principles of the 'SPALL' criteria, defined by King and Ecob[6] and listed below.

Structural: Possesses the required structural properties.

Protection: Provides protection for the reinforcement.

Application: Can be applied effectively within the given constraints.

Longevity: Once applied it remains in place.

Looks: Has an appropriate surface finish.

The 'SPALL' parameters, in combination with assessment of the quality of the processes of selection and specification of the repair, have been reviewed at individual sites. Site observations and testing have been used to gather information on the effectiveness of the technical, site activity aspects of the repair, i.e. the physical actions involved in execution of the repair. The contract information and records have been used to evaluate the process effectiveness, i.e. the planning and management of the repair.

1.5 Structure

The project was carried out in four main stages. The first was a data gathering phase in which the literature available for the types, methods, standards, investigation techniques and performance of repairs was reviewed[7]. This confirmed there is little independent data on the long-term *in situ* performance of most concrete repair systems. The second stage involved identifying the sites to be visited, contacting the owners/operators, sourcing record documents, and executing risk assessments and designing the investigation procedures. The third stage involved visiting 45 sites to form a database of defects[8]. Specialist techniques were used to investigate a large number of repairs and CP systems at one major site[9]. The final and current stage of the project involves the analysis and presentation of the data[10]. All of the reports will be available via the HSE in 2002.

2. Site investigation

2.1 Number and type of sites investigated

Patch repairs were targeted where the depth of repair was less than the full depth of the element, typically less than 100mm deep, and the areas repaired were typically less than 1m x 1m. A wide variety of repairs and repaired structures were included to provide a representative population for study. In total, 45 sites were visited, and have been described in detail in relation to age, natural and service environment, repair history and condition. However, the sites remain anonymous. At certain sites, there was more than one type, generation or condition of repair, and in total 65 locations were examined.

The sites were located in throughout England and included infrastructure, public buildings and nuclear facilities in coastal, estuarine, river and inland locations. Repaired elements included beams, columns, slabs and walls from structures including bridges, tunnels, power stations and other reinforced concrete frame buildings. The type and environment of structures visited is summarised in Table 2 and Table 3.

Table 2 Types of repair examined

<table>
<tr><td rowspan="3">Structure type and location</td><td colspan="4">Concrete frame structure</td><td>Car park</td><td colspan="3">Road bridge/viaduct</td><td>Road tunnel</td><td>Others</td></tr>
<tr><td>Public</td><td colspan="3">Industrial</td><td rowspan="2">Inland</td><td rowspan="2">Inland</td><td rowspan="2">River</td><td rowspan="2">Estuarine</td><td rowspan="2">Estuary</td><td rowspan="2">Inland underpass</td></tr>
<tr><td>Inland</td><td>Inland</td><td>Estuarine</td><td>Coastal</td></tr>
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<tr><td>Total</td><td colspan="4">14</td><td>5</td><td colspan="3">20</td><td>5</td><td>1</td></tr>
</table>

Table 3 Types of repair examined

Type of repair	Hand/trowel applied			Sprayed		Flowable	CP	Crack Sealing
	proprietary		conventional	proprietary	conventional			
	dense	high build						
Total	25	8	7	9	3	9	5	3
Total	40			12		9	5	3

At 48 locations the repairs were investigated through visual inspection, hammer surveying, non-destructive testing, and intrusive sampling, and 60 core samples were examined in the laboratory. At 13 locations the repairs were examined by visual inspection and hammer surveying. At 3 further locations visual inspection was supplemented with non-destructive testing (NDT).

The sites were selected where safe access could be achieved to repairs where some knowledge or documentation of repair age, locations, type and method existed. The repair locations were selected to represent a range of sizes, types and application methods. The repairs were applied by hand or trowel, spray, or flowable methods. Mostly the repairs were of cement-based conventional materials or proprietary repair systems, and up to 12 years old. Seven different CP systems were also examined at four different structures. Each was associated with repair with cementitious materials prior to installation of the CP system.

2.2 Records of repair

For each site, an attempt was made to discover the maintenance history, the cause of deterioration, and the parties involved in the repair process. This involved consultation with the structure owner/operator and recovery of documents related to condition prior to repair, the cause of defects, the repair contract, specification and method statement, and detailed description and/or data sheets for the materials used in the repair. Photographs of the repair contract were also invaluable.

The level of documentation available varied greatly. The older the repairs, the more difficult it was to recover all of the information. There was also a difficulty in recovering documents where the ownership of the structure had changed. Where more than one repair generation occurred, there was often no information relating to the earlier repair phases.

For a number of sites no records were available, or records of repair did not appear in existing engineering and maintenance files. Records from Health and Safety files did not contribute significantly to the information recovered for the sites, despite a requirement under CDM regulations[11] for retention of the Health and Safety file for the life of the structure.

2.3 Road Tunnel Deck Study

The research also took advantage of replacement of the reinforced concrete deck of a major UK road tunnel during 2000-2001. Sections of the deck containing repairs and CP systems were inspected and sampled. A total of 26 cores and 18 sections measuring approximately 600mm x 500mm, were sawn through the full depth of the deck. The sections were water jetted to expose a 250mm length of each bar in one face, and pull-out testing was carried out on 57 bars at the University of Birmingham.

3. Site investigation methods

The performance of the repairs was assessed through a combination of visual inspection, non-destructive testing e.g. hammer tapping, half-cell potentials, cover meter survey, carbonation depth. Destructive sampling and testing was carried out where permitted by the structure owner, and included pull-off resistance, extraction of a core for petrographic examination and electron microscopy, assessment of carbonation depth, and sampling for chloride and alkali contents. Other investigation techniques, such as chloride and alkali determinations were used at specific sites where deemed necessary. The sample holes were repaired with a proprietary high-build repair system and the site cleared and vacated.

Where possible, more than one repair was sampled, identifying contrasts in condition, appearance, age, location, and exposure conditions.

3.1 Effectiveness of investigations

Visual inspection and hammer tapping were of greatest value on site, particularly when combined with core sampling or break out. The visual inspection provided a rapid assessment of the overall condition of the structure, its environment, and condition of repairs and coatings. Inspection of defects, particularly when combined with documentation of the condition and maintenance history of the structure, provided an insight into the causes, severity and timing of deterioration. In comparison, the value of the half-cell survey and pull-off resistance tests were limited.

3.2 Half-cell surveys

Half-cell surveys were carried out at many locations and most required careful interpretation. In general, there was a marked difference in potential values for the repairs and for the substrate concrete. Typically the repairs had a less negative value, and was often positive, indicating a dry substrate. A common trend found in columns and walls, in both repairs and substrate, was of increasing negative potential towards ground level, and particularly within 0.5m of ground level. This is associated with the increase in moisture content of the substrate and not necessarily associated with corrosion activity. However, in the road and car park structures, chloride concentrations may be higher in the lower portions of elements exposed to salt spray. This can result in higher corrosion potentials. The portions within 1.5m of ground level exposed to spray were observed to have a high incidence of deterioration and repair.

Where reinforcement was exposed at the surface, either through low cover or through spalling of the cover concrete, an increase in negative potential was typically found only by detailed mapping and often over a very limited area. This was probably related to the general lack of moisture within the dense repair materials. Consistently high negative potentials were found only in wet substrates, for example beneath leaking joints.

Previous research[12] has found the surface zone of cementitious systems to have significant effects on oxygen diffusion and resistivity. These ‘skin effects’ can interfere with half-cell surveys and provide an inaccurate impression of corrosion potential at depth.

3.3 Pull-off resistance

The pull-off testing was characteristically problematic and time consuming particularly on overhead locations and on rough substrates. Failure typically occurred at the interface between repair material and the substrate (36%), within the concrete substrate (21%) or in the adhesive (25%). The recorded failure values typically ranged between 0.1 and 0.9N/mm^2, with a mean value was 0.48 N/mm^2 and standard deviation of 0.22 N/mm^2.

No difference was found between pull-off resistance in repairs with substrates prepared by waterjetting compared to those prepared by mechanical break out. However, other research has demonstrated significantly higher bond strength on overlays applied to water jetted substrates compared with sand blasted and mechanically broken substrates[13].

3.4 Core sampling and petrographic examination

Examination of the core samples and core holes provided information on macroscopic feature such as void content and distribution, discontinuities, cracks, condition of reinforcement, and configuration of repairs and interfaces. The petrographic examination of the repair and substrate provided detailed information on the microstructure and quality of the repair material, substrate and the interface between them. This identified differences and subtleties in repair composition and structure that were not appreciated on site, such as the extent and depth of fine cracking and microcracking from the external surface, the depth of carbonation along these features, layering in repairs and subtle differences in composition of the binder.

4. Preliminary findings

4.1 Reason for repair

The majority of the repairs examined related to corrosion of the reinforcement embedded in the original concrete substrate. Overall, approximately 60% of the locations examined had deteriorated, or were repaired, as a result of chloride induced corrosion or the potential for it. Approximately 25% had deteriorated as a result of carbonation induced corrosion, and the remainder by drying shrinkage or other mechanisms. The structures built between the 1930’s and 1950’s had mostly required repair as a result of carbonation induced corrosion. These were almost exclusively in locations devoid of an external source of chlorides. In most structures there was evidence of some post-repair deterioration. This was often associated with the original cause of deterioration, such as chloride ingress. Performance of the repairs was not always a measure of the effectiveness of the management strategy.

4.2 Repair structure

Many features of the repairs could be identified, such as sawn perimeters, feathered edges, different methods and depths of break-out, presence of bond coats, reinforcement primers, levelling coats, external coatings, and the occurrence of layers, partings, cracks and voids. The components and procedures described in the repair records did not always match those found in the site investigation. The thickness of applied layers in hand or trowel applied materials was often greater than that recommended in the materials data sheets. There was little evidence that this had compromised repair effectiveness.

Many of the repairs were finished flush with the surrounding surfaces, resulting in reinstatement of low cover depths, sometimes less than 10mm. Repairs at several locations had an intentional local increase in cover to overcome this. At other sites, the cover was enhanced by application of cementitious render or additional protection was afforded by a paint or high performance coating at the external surface.

4.3 Cracking in repairs

Cracking occurred at the external surface in approximately 60% of repair locations examined. Approximately 45% of repair locations contained surface crazing and/or cracks resulting from drying shrinkage. The cracks commonly passed from the external surface of the repair to the embedded reinforcing steel, and can pass through the full thickness of the repair. The drying shrinkage cracks are typically less than 0.1mm wide, and the fine cracks grade into microcracks, clearly visible in the repairs under petrographic examination, and remain readily resolved at 0.005mm width.

There is evidence of penetration of water into cracks and carbonation of the binder adjacent to cracks. At approximately 20% of repair locations the depth of carbonation along the cracks was greater than the minimum cover depth at the site or the depth of reinforcement intersected in the core sample. Fine cracks and microcracks in repairs less than a year old were carbonated to depths of up to 21mm. The depth of carbonation from the crack surfaces was very limited and typically no more than 2-5 times the width of the crack.

Cracks were also found at the margins of repairs. The extent of carbonation in the repair material was mostly insignificant, but the substrate concrete was commonly more porous than the repair, and carbonated to greater depths. This is significant where reinforcement bars pass from the substrate into the repair and local depassivation at the steel could result in corrosion. The ingress of chlorides into cracks is also known to result in corrosion of reinforcement at the perimeter of repairs[14].

4.4 Deterioration of repairs

Deterioration had occurred to some ‘holding’ repairs in aggressive environments. A small proportion of ‘long-term’ repairs were failing prematurely. The cause of deterioration was mostly site-specific and included reoccurrence of the original cause of deterioration, ingress of water and chloride, shortcomings in the specification, crazing or cracking in the repair material, carbonation of poor quality material, vibration in the structure, reinstatement of low cover and the presence of a cavity at the repair/substrate interface. Repairs were ineffective mostly in the ‘Protection’, ‘Longevity’ and ‘Looks’ aspects of the SPALL criteria.

Evidence for incipient anode formation was found at several sites with cracking or spalling of the repair and/or surrounding concrete, typically at or close to the repair perimeter. All of these locations had been repaired because of chloride-induced corrosion. There were also locations where corrosion had continued at the repair site, not necessarily through incipient anode formation, but mostly through ongoing exposure to saline water, or the presence of carbonated concrete adjacent to the repair. The repair planning and management processes for these sites were not fully effective.

4.5 Performance of CP

Each of the sites where CP had been operational showed very low levels of deterioration to the substrate concrete and patch repairs. Some deterioration of the external coating systems was noted. The mean bond stress determined through pull-out testing on 57 bars was $3N/mm^2$. The pull-out testing found no evidence that CP had affected the bond strength for plain round bars in original deck concrete or the sprayed concrete repairs in areas with CP and in areas without CP.

Examination of core samples confirmed that there was no evidence of significant corrosion of the bars embedded in concrete protected with CP systems. Ionic mapping of the binder in repairs and original concrete confirmed trends of ionic concentrations at the surface anodes and embedded cathodic steel reinforcement. No evidence was found for significant deterioration resulting from these concentrations.

5. Preliminary Conclusions

This project aims to define and assess the effectiveness of *in situ* patch repairs and CP systems. A large population of repairs of up to 12 years age have been investigated.

Records for the repair contracts have been examined but are not always complete or available.

The majority of structures had repairs to corroding reinforcement, caused predominantly by chloride contamination. Many of the repaired structures showed evidence of post-repair deterioration, particularly where the cause of deterioration was not effectively treated. However, the structures with CP systems were in notably good condition.

Many of the patch repairs contain shrinkage cracks which represent a performance limiting factor.

6. Acknowledgements

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Industry Management Committee.

Institution of Civil Engineers Research and Development Enabling Fund.

MM Group Research and Development Fund.

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Detect and repair of defects on the confinement structure at Paks NPP

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ABSTRACT

Paks Nuclear Power Plant is the only commercial nuclear facility in Hungary, which has been operational since 1982. Like other N-plants, Paks NPP is also exposed to major challenges due to plant aging and changes in circumstances that affect the operation. The defense in depth concept is a corner of nuclear safety, therefore the confinement concrete structure and its aging phenomena is on the focus of the utilities. Declaring the lifetime extension program, Paks NPP pays distinguished attention on confinement integrity and achieved significant results on this field.

Content of presentation

1. Introduction
2. The construction and structure of hermetic space (confinement.)
3. Methods of examination of leakage and strength test
4. The results of leakage tests.
5. Repairing of defects found during examinations.
6. Short review of repair of the liner and concrete structure at the Unit 1. of NPP Paks.
7. Conclusions.

1. Introduction

The third element of the defence in depth concept is the confinement which provides the last physical barrier against activity release to the environment.

Therefore, the leakage of the confinement is strictly limited, which is one of the fundamental conditions of plant operation.

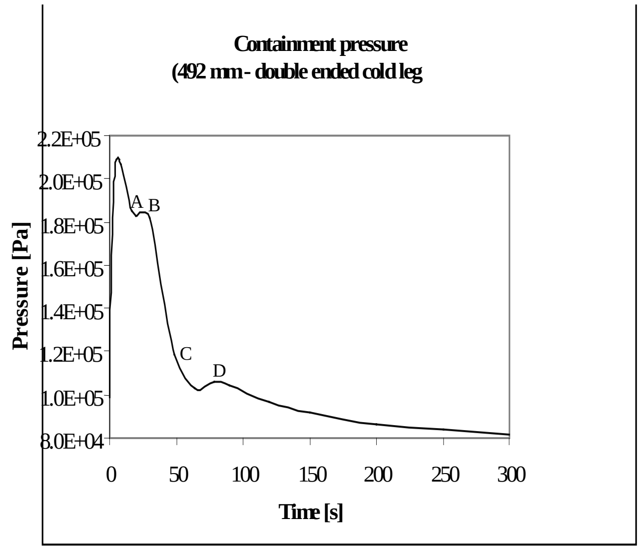
To assess the integrity of the confinement, the utility has to determine the leak rate of the confinement, even there has been a locally uncontrollable hole opened on the liner. Outage for maintenance and refuelling is a typical case.

2. The construction and structure of confinement

Net volume of confinement is 50 000 ÷ 54 000 m^3.
Lower stage is –6.5 m-, higher stage is +40.5 m.

Carbon steel liner is on the inner side of confinement’s rooms, and outer side of reactor compartment walls.
Steel liner’s thickness is 8 mm.
Thickness of concrete is 800÷1500 mm.

Pressure transient at a LB LOCA DBA at VVER-440 confinement.



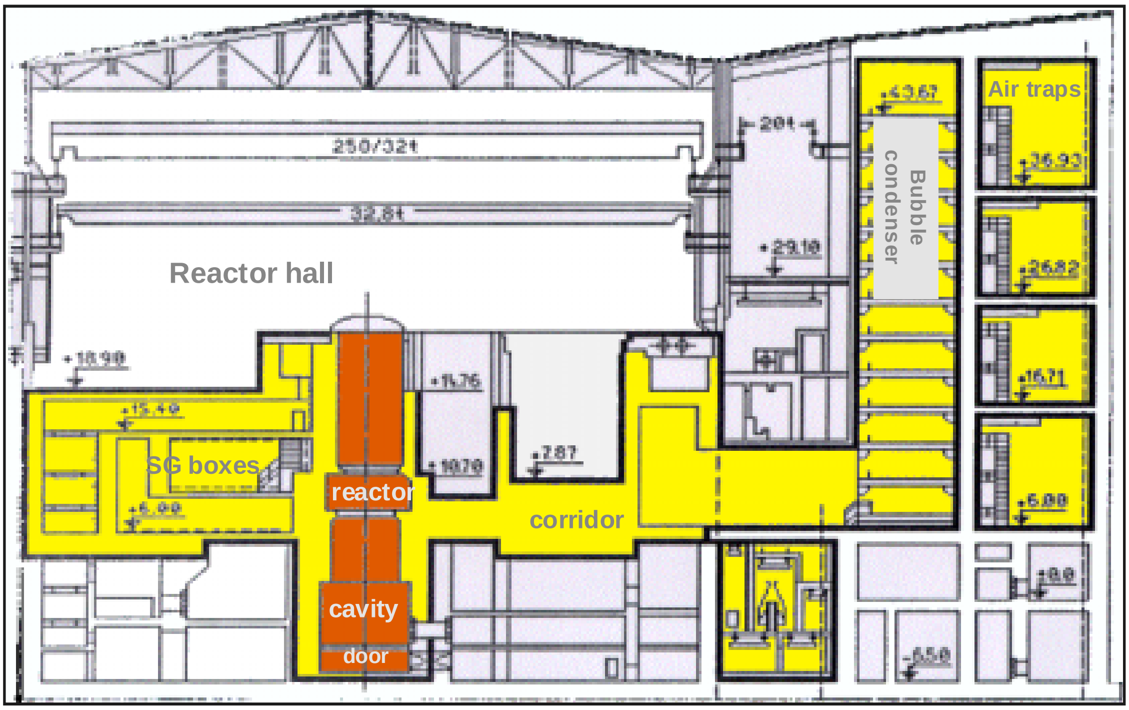
A [16. s] - HPSI started

B [30. s] - Water spill back from the trays started

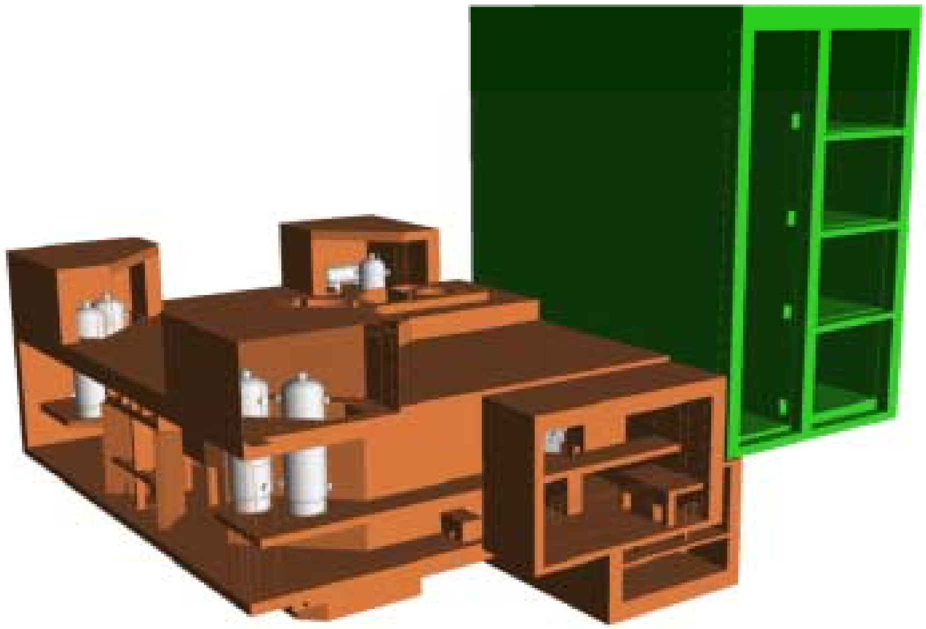
C [48. s] - Water spill back from the trays finished (90 % of water volume spilled back)

D [86. s] - spray injection started

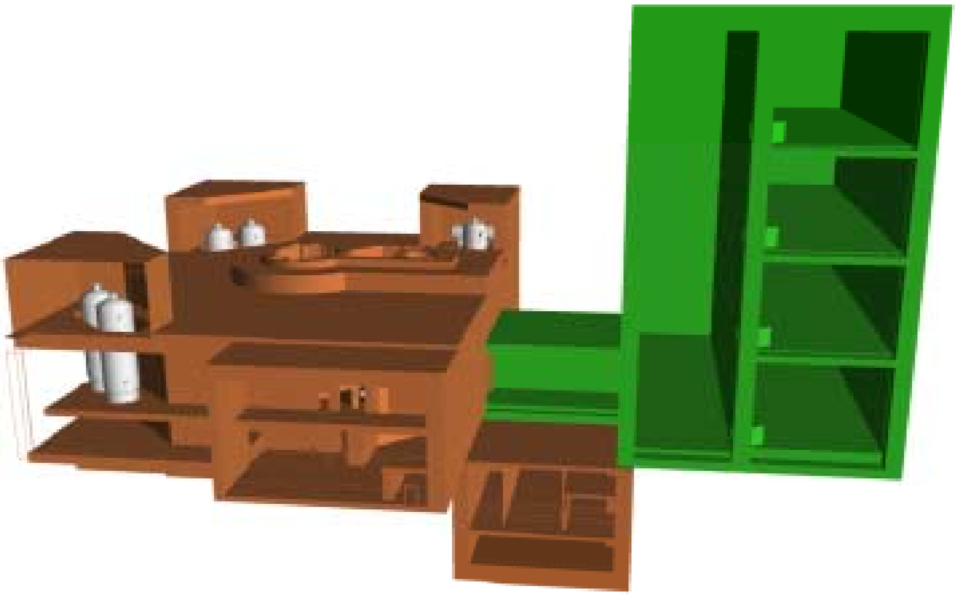
Confinement building with the Bubble Condenser



3-D modell of confinemet



Comfinement's 3-D modell

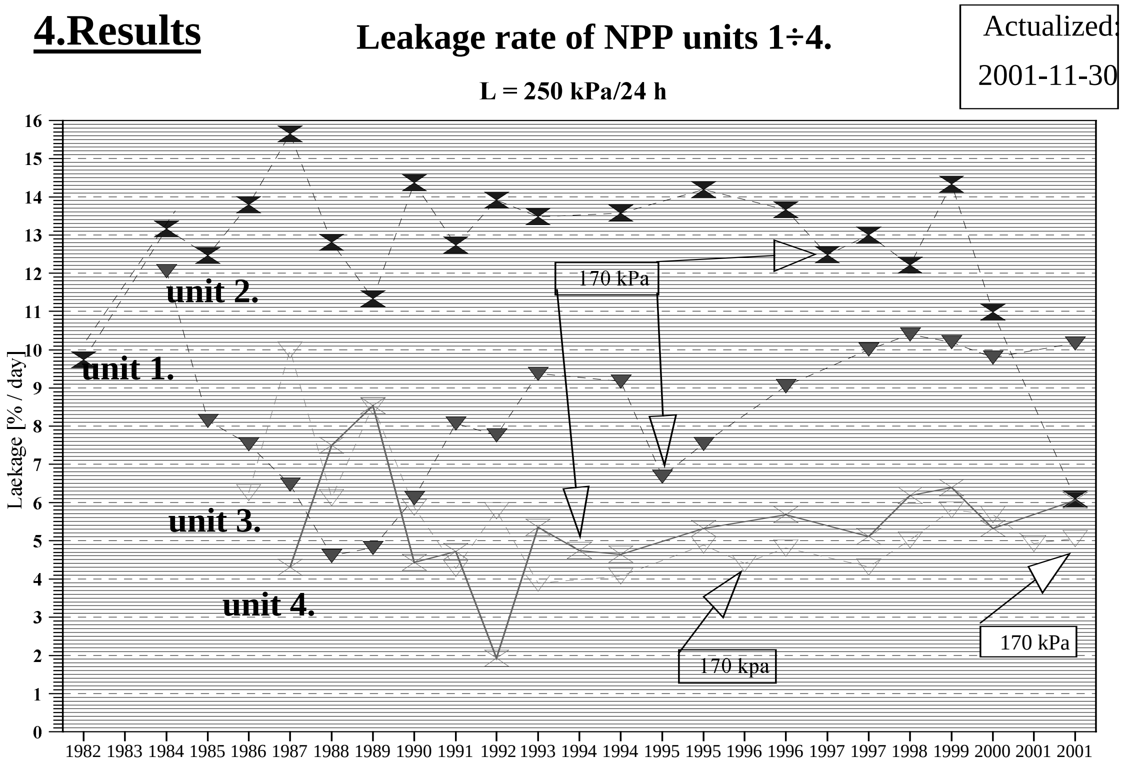


3. Methods of examination of leakage and strength test

We had carried out a leakage test in full designed presssure - 250 kPa - at first start-up phase. We had measured the leakage of confinement at three gaugh level;
120, 170 and 250 kPa.
We had measured deformation of walls at same time.

Repeted leakage tests are achieved on 120 kPa.
Leakage rate is converted to 24 hours and 250 kPa.
We had managed a leakage test at 170 kPa on each unit at 1994÷1997.
We use the two point method, while pressure drops.

4.Results


Leakage rate of NPP units 1÷4.
L = 250 kPa/24 h
Actualized:
2001-11-30
Laekage [% / day]
unit 1.
unit 2.
unit 3.
unit 4.
170 kPa
170 kpa
170 kPa
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
1982 1983 1984 1985 1986 1987 1988 1989 1990 1991 1992 1993 1994 1994 1995 1995 1996 1996 1997 1997 1998 1999 2000 2001 2001

5. Repairing and checking defects found during examinations

Defects of kiner
- welding→ local test
- injection plastic material → integrated leakage test

Defects of hermetical closures
- exchanging gasket material → local test
- exchange sealing equipments → integrated leakage test
- making new gasket construction → integrated leakage test
- resealing of door'honges → integrated leakage test

6. Short review of repair of the liner and concrete structure at the Unit 1. of NPP Paks

We have made a program improve hermetization of unit 1. from 1998 co-operating with company VÚEZ a.s. Slovakia.
Mean activities:

- searching defaults during ILRT and depression test
- injecting the space between liner and concrete
- measuring alteration of pressure gaugh under liner while carried out an ILRT
- decomposition of concrete to liner at blowdown air corridor and injecting
- decompositioning of concrete at a hermetic room explorating a connection to liner and repairing it

Injektion point No 1./19 deconstructed concrete in A201/1

Injektion point No 1./19 with injection cap

Injektion point No 1./19 with local control cap

Excaveted connection to liner in room A306/1

Excaveted connection to liner in room A306/1

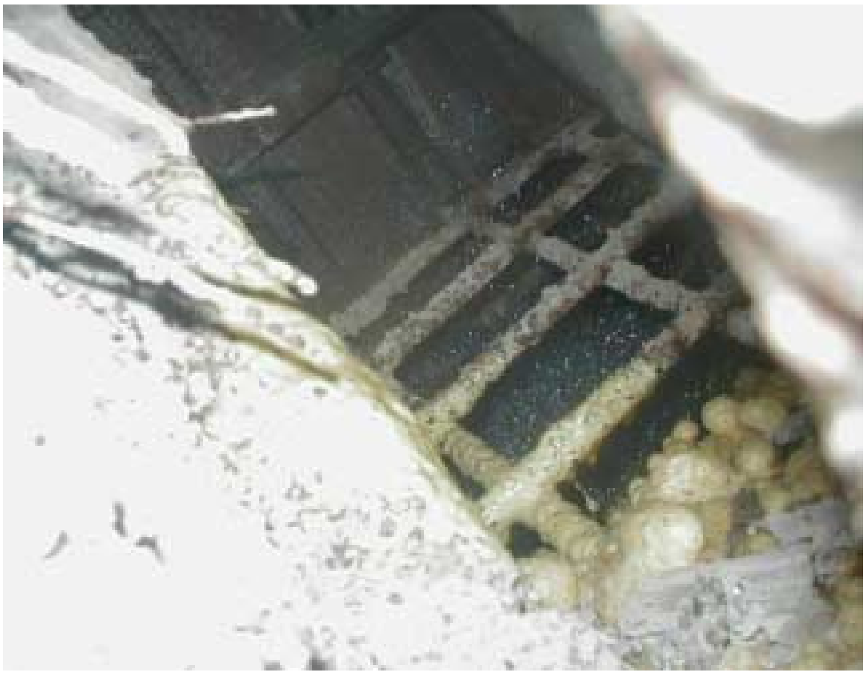
Repaired connection in room A306/1

Excaveted connection in room A306/1

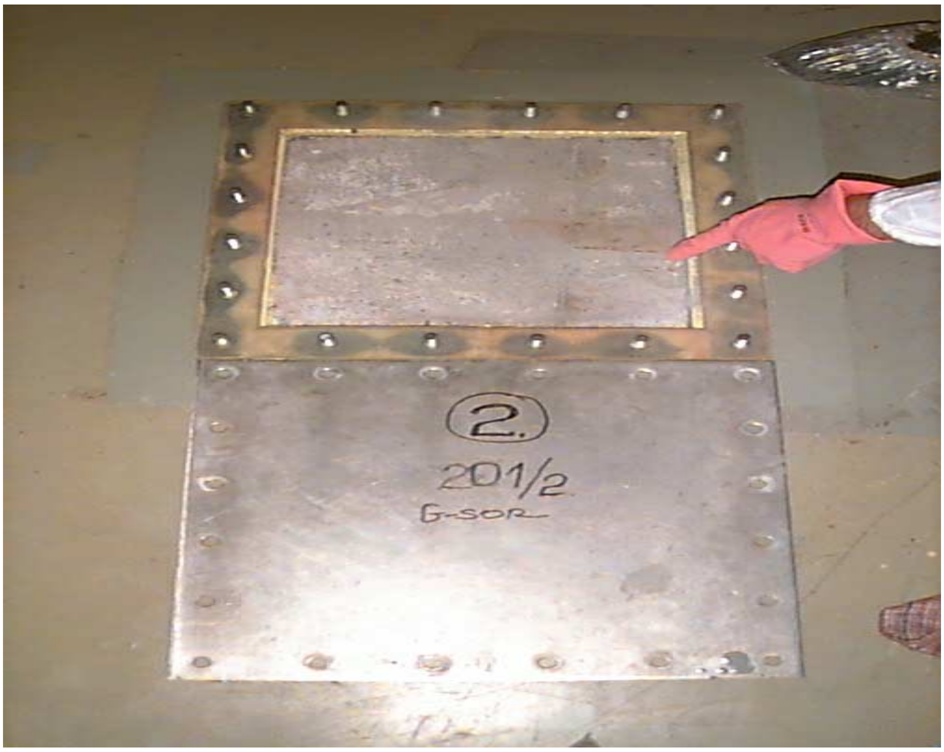
Repaired connection in room A306/1

Hole in structural concrete discovered by drilling for injektion

Inside of hole in structural concrete
discovered by drilling for injektion



Checking hole No 1.for borical leakage and concrete at unit 2



Checking hole No2. for borical leakage and concrete at unit 2

Checking hole No3. for borical leakage and concrete at unit 2

Checking hole No1. for borical leakage and concrete at unit 3

Checking hole No2. for borical leakage and concrete at unit 3

Checking hole No 3. - unit 3

Other activities to repair estate of concrete:

- gap of concrete between the spent fuel storage pool and №1. pool
- destructive and chemical disquisition of concrete samples
- model supported mechanical and chemical experiments have been performed
- long term program to check the status of concrete structure
- evaluation of concrete structure for lifetime extension

7. Conclusions

7.1. Notable degradation of concrete hardness has not been found.

7.2. Surface corrosion has been found on the metal structure in the concrete

7.3. The confinement leak rate is within the limits of Techn. Spec.

7.4. Leak tightness enhancement program on the unit 1 confinement is in progress, significant results are achieved.

SESSION A: OPERATIONAL EXPERIENCE (continued)
Chairman: Dr. James Costello, USNRC (USA)

Steam Generator Replacement At Ringhals 3 Containment, Transport Opening

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Abstracts

At the steam generator replacement at Ringhals 3 1995 an opening 6 x 8 m was taken in the cylindrical containment wall about 12 m over the ground level. The wall is build up of an outer prestressed concrete wall, steel liner and an inner concrete wall. The prestressing reinforcement in the outer concrete wall consists of horizontal and vertical tendons. Each tendon was prestressed to about 5000 kN from the beginning. The tendon ducts were filled with grease.

Before the opening was taken a temporary wall with a gate was build on the inside of the containment wall. When starting making the opening, the tendons within the opening were de-tensioned and pulled out. A additional number of horizontal tendons under and over the opening were de-tensioned. The opening was cut through drilling 300 mm holes along the side of the opening into the steel plate from both outside and inside. Then the steel plate was cut and the wall plug removed.

During the transport of the steam generators the work with the restoring of the wall started. The concrete surface was prepared and reinforcements bars were drilled into the old concrete. When the transport was finished the steel plate was restored, tendon ducts joined and the reinforcement put in place. To prevent early cracking in the concrete tubes for cooling water were mounted in the form. The inner form was casted at first and then the outer one.

After that the concrete has reached sufficient strength the tendons were mounted in the ducts and tensioned to the same force as before de-tensioning. After the tensioning the ducts were filled up with grease again.

A special inspection program was performed to see if there were any degradation in the concrete, the tendons or the steel plate. The result was that we could see no degradation on the steel plate or the tendons. The measured tendon forces are higher than calculated. The strength of the concrete had reached a medium level of 91 MPa (Original level 50 MPa). The carbonisation had reached 8 – 10 mm into the concrete. Our conclusions are that we have not seen any degradation which is a threat to the containment in the foreseeable time.

1. Introduction

1.1 Background

In 1992 Ringhals decided to change steam generators at Ringhals 3. The degradation of the tubes had gone far and the capacity was reduced to 88 % since 1988. A steam generator replacement had been done at Ringhals 2 during 1989 and the experiences from the replacement were good.

The planing of the replacement started during 1992, and KWU was chosen to manufacture the steam generators. For the replacement at Ringhals a consortium consisting of Siemens and Framatome was engaged. The civil works were carried out of a subcontractor, NCC, to the consortium. NCC is one of the biggest contractors in the Swedish building industry. The replacement work started in the beginning of June 1995 and was finished within 90 days.

1.2 Transport procedure

The steam generators were transported to Ringhals by boat to the harbour at Videbergshamn near Ringhals Site. From the harbour the steam generators were transported by a special vehicle to a storing place within the Ringhals area. Some preparation works were done on the steam generators. The replacement procedure started with making the opening. As soon as the opening were ready the old steam generators were taken out from the containment one at a time. The steam generator was lifted up from its position by the polar crane and laid down on trolleys on girders at level +115. Then the steam generator was transported through the containment opening and lifted down to ground level by a lifting device. The vehicle then took the steam generator to a storing place. When the three old steam generators were taken out it was time for the new ones to be transported into the containment. The transport procedure was the same in the opposite direction.

1.3 Description of the wall

The cylindrical containment wall consists of one outer concrete wall, a steel liner and an inner concrete wall. The inner diameter of the containment is 35.4 m and the height is 52 m.

The inner concrete wall is 0.33 m thick concrete reinforced with a grid of Ø16 mm rebars distance 200 mm in the inside. The compressive strength of the concrete in the wall is K50.

The steel liner is 7 mm thick. On the outside of each joint between plates in the steel liner there is U-profile welded in order to make it possible to see if there is any leakage through the joint. The canal inside the U-profile is connected to each other in special sections. From each section there is a pipe drawn to a gallery where it is possible to test each section. To the steel liner there are profiles welded to which the tendon ducts are fixed.

The outer concrete wall is 0.77 m thick concrete reinforced with a grid of Ø 25 mm rebars distance 200 mm in the outside. The covering concrete layer is 50 mm. The compressive strength of the concrete is K50. About 0.3 m in the wall there are horizontal tendons and about 0.5 m in the wall there are vertical tendons. The distance between the vertical tendons is 0.75 m and the average distance between the horizontal tendons 0.4 m.

Both vertical and horizontal tendons consists of 139 Ø6 mm wires. The tension load is just below 5 MN. Each horizontal tendon stretches half a turn round the containment. The total number of horizontal tendons is 245 and vertical tendons 153.

1.4 Preparation work

In order to prepare the staff that were going to do the civil some practice was done on the wall block from Ringhals 2. The containment wall in Ringhals 2 has the same construction as it has in Ringhals 3. On the wall block the staff trained seem drilling and other work procedures in the same manner as it was planned to be done at Ringhals 3.

2. The transfer opening

2.1 Protective wall inside containment

The work inside the containment started as soon as the plant was shut down.

After erecting scaffolding and cleaning the inside of the containment wall and the floor next to the transfer opening, a protective wall was installed. The floor between the protective wall and the containment wall was covered with steel plates. The protective wall had a steel structure and covered with steel panels. There also was a ceiling of steel panels built between the protective wall and the containment wall.

In the protective wall there was a MEGA-door installed through which the steam generators were transported.

2.2 Detensioning and removal of the tendons

Within the opening there were nine vertical tendons and 20 horizontal tendons which were detensioned and dismounted. Another 56 horizontal tendons were detensioned. Theese tendons are situated below, above and on the other side of the containment than the opening. The work with the tendons started as soon as the plant was shut down.

The detensioning work started with erecting of scaffolding outside the containment at the pilasters. Also for preparation the jacks and x/y-writers were calibrated.

For the tendons the grease cans were dismounted and the anchor parts were cleaned from grease. The jack was coupled to tendon top end for the vertical tendons and at both ends for the horizontal. The present tension load was checked. Then the tendon was tensioned to a load of 5 MN and the extension recorded. The detensioning of the tendon was done in two steps and the contraction recorded. The jack was removed.

Then the tendons were dismounted through special procedures, one for the vertical tendon and one for the horizontal. The tendons were cleaned from unnecessary grease, inspected and winded up on a hydraulic winch.

The tendons and grease cans were stored indoors during steam generator replacement.

2.3 Cutting of the opening

The cutting of the opening 8 m x 6.6 m started as soon as the tendons in one side of the opening was removed. The first operation was to drill holes with diameter Ø 300 c 200 from both outside and inside at the same time. Totally six drilling units worked parallel. The holes were drilled almost into the steel liner. The concrete near the steel liner was chipped away. In the bottom of the opening a hydraulic jack were mounted and special holes were made in the corners for the sliding beams which should be used for removal of the wall block. Some drilling for rebars and bolts were done for the supports for the sliding beams and to fix the inner concrete plug to the outer.

When the drilling was done and the fuel elements were removed from containment the steel liner was cut. In the bottom corners of the block holes were cut in order to make it possible to place the sliding beams. A sleigh was mounted on the sliding beams and the wall block was lowered down on the sleigh. And before the last pieces of the liner was cut the wall block was secured to the sleigh.

The block which weighed about 125 tons was drawn out from the containment wall, then lifted down to the ground level and transported to a storage area.

2.4 Preparing surfaces

After removal of the wall block the work with preparing the concrete surfaces took place. Shear recesses were cut in the concrete surfaces in order to transfer shear forces in the joint. In the surface at the top of the opening there were ventilation channels cut out to make it possible to fill up the form with concrete properly.

The tendon ducts were cut at the level of the concrete surface and the new tendon ducts were joint with inserts.

The concrete around the test channels for the steel liner was cut out.

To replace the rebars that were cut, new rebars were drilled in the concrete near the outside respectively the inside. A new layer of rebars was installed between the vertical and the horizontal tendons. The drilled holes were up to 2500 mm deep. Rebars with joint nuts were grouted in the drilled holes according to a special procedure.

This work were done during the period when the transportation of the steam generators went on through the opening.

2.5 Restoring of the steel plate

The work with preparing the edge of the steel liner went on during the period of transportation. A new prefabricated steel liner was placed in the opening. The new steel liner was connected to the old steel liner with two small steel plates one on the inside and one on the outside. The channel between the small steel plates was connected to the test channels for the old steel liner. The test channels that were cut off, when drilling the opening, were restored. Attachments for the tendon ducts had been welded to the steel liner at the prefabrication.

2.6 Restoring the inner part of the containment wall

The first work sequence to be done was to mount all the connecting rebars. Then the vertical and the horizontal rebars up to 1,5 m were mounted. The first part of the formwork was mounted and the first part of the casting could start up to 10 cm below the edge of the formwork. The working procedure was repeated with mounting the horizontal rebar, mounting next part of the formwork and casting for each 80 cm.

In the last section there were some special arrangements done to secure that the opening was properly filled with concrete. A pump was connected to the formwork and the concrete was pumped into the formwork until it came out in the ventilation opening at the top of the opening. The formwork was made of steel plates and left after the work was finished.

2.7 Restoring the outer part of the containment wall

In order to secure a low temperature in the concrete during the hardening pipes for cooling water were mounted at three depths in the concrete. The first row of cooling pipes was mounted close to the steel liner, the second at approximately the same depth as the vertical tendons and the third just outside the horizontal tendons.

The work procedure had the following parts:

- Mounting the first row of cooling pipes.
- Mounting vertical tendons ducts to the attachments and connecting with the existing ends of the tendons ducts.
- Connecting and mounting the first layer of vertical rebars.
- Connecting and mounting the first layer of horizontal rebars.
- Mounting the second row of cooling pipes.
- Mounting horizontal tendons ducts to the attachments and connecting with the existing ends of the tendons ducts.
- Mounting the third row of cooling pipes.
- Connecting all the vertical rebars and horizontal up to 1,5 m in the second layer.
- Mounting all the vertical rebars and the horizontal up to 1,5 m.
- The first section of the formwork was erected.
- After connecting the cooling system to the cooling pipes the casting took place up to 10 cm below the top of the formwork.
- The work procedure with connecting and mounting horizontal rebars, erecting formwork and casting was repeated each 80 cm.
- To the last section of the formwork a pump was connected and the concrete was pumped into the formwork until it came out in the ventilation opening at the top of the opening.

The concrete quality used was K50 with the temperature 5 ºC. In order to prevent cracks from high temperature during the hardening the concrete was cooled. The incoming temperature of the cooling water was to 2 ºC. The temperature of the cooling water coming out of the pipes was registered and compared with calculated values. The supervision of the temperature went on for some time after that the casting was ready and the temperature had decreased to a normal level.

2.8 Tensioning of the tendons

The work with the mounting and tensioning of the tendons started about 12 hours after the casting was finished. The first moment was to insert the tendons that are crossing the opening in the ducts. When the tendons had got through the duct the anchors were thread on each end and a button head was made at each wire.

Before the tensioning work started the jacks and x/y-writers were calibrated. When the compressive strength in the concrete in the opening had reached 36 MPa the tensioning of the tendons outside the opening could start. To start with the tensioning of the tendons crossing the opening the compressive strength in the concrete must have reached 40 MPa.

The tensioning was done according to a specified procedure to the specified load and the same shims were placed at both ends as before detensioning. Both the load and the elongation was recorded. When all the tendon were tensioned the grease cans were mounted the cans and tendon ducts were injected with grease.

One incident happened when the horizontal tendons were inserted in the tendon ducts. One of the ducts was damaged so that the plate in the duct was pressed to a stop in the duct. It wasn't possible to insert the tendon. The place, where the stop occurred, was located. It was situated outside the opening. It was necessary to cut the concrete away to uncover the stop. It was done with water jetting. The stop was found and the duct repaired and the concrete restored. The tendon was then inserted and tensioned.

2.9 Removing of temporary constructions

A couple of days after the casting of the concrete the removal of the temporary wall on the inside begun. The area was cleaned and the material was transported out of the containment.

When the tensioning of the tendons was finished the scaffolding was removed.

3. Inspection program

3.1 Contractors control program

In the scope that the contractor had there was prescribed a control program for the civil work that included normal control on concrete works, the drilling, cutting, welding and prefabricating the steel liner and the tendon works. A special control plan was drawn up for this purpose.

3.2 Status of the containment wall

At some incidents that had happened at other plants before, questions about the status of the containment wall had been raised. In order to answer some of these questions a control program was established. The main parts in the program were:

- Visual inspection of all uncovered concrete surfaces to look for cracks, cavities from the casting and other abnormalities.
- Visual inspection of uncovered rebars to look for corrosion and how the concrete has enclosed the rebars.
- Visual inspection of the steel liner to look for corrosion.

Take out concrete cores to examine the compressive strength, the carbonation deep and the deep of the chloride penetration.

3.3 Results

One observation made, was a crack between the steel liner and the concrete at the inside of the steel liner. This phenomenon is known before from Ringhals 2 at the steam generator replacement. The crack width was approximately 1 – 2 mm and according to a special investigation the crack will remain in the area next to the opening but disappear for the rest of the containment wall at the tensioning of the tendons.

There also was one small crack in the outer wall at the top of the opening. The crack width was less than 1 mm and was probably caused by shrinkage of the concrete. The crack closed probably at the tensioning of the tendons. No other signs of degradation of the concrete were discovered.

There were no signs of corrosion on the rebars and the concrete had enclosed the rebars quite good.

The uncovered parts of the steel liner showed no sign of corrosion. The steel liner surface had a thin layer of cement mortar.

The compressive strength was 91 MPa as an average value. The origin compressive strength was set to 50 MPa and samples taken by the casting showed values of 60 - 70 MPa after 91 days. There is an increase of the compressive strength with increasing age of the construction.

A normal value of the carbonation was 8 – 10 mm. It is about 20 years since the containment wall was poured. Our estimate is that when the construction reach 100 years of age the carbonation have reached a depth of about 20 mm in the wall.

The result from the test of chloride gave that the concentration of chlorides is very low. The highest value was 0,2% chloride of the cement weight at the surface and less than 0,1% in depths of 3 – 6 cm.

The conclusion of the inspections and tests mentioned above are that the containment wall is in good condition and there are nothing that indicates a degradation of the containment.

In Service Inspection Programme and Long-term Monitoring of Temelin NPP Containment Structures

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Abstract

The paper describes monitoring systems of Temelin containment concrete structures and pre-stressing systems. Reliable information on the actual state of containment structure is necessary for condition assessment as well as for detection of local defects and effects of ageing. In service inspection programme as a main tool of preventing structural defects and damages will be discussed. Results of Temelin containment inspections and tests will be presented.

Containment structure

The Temelin NPP is formed by two VVER 1000 MW units. At present the start-up of the 1st unit at 100% power output is underway, for the 2nd unit fuel has been loaded and the physical start-up process is underway. The units of the VVER 1000 MW type have a PWR (pressurised water reactor) reactor and a containment of pre-stressed reinforced concrete. Typical cross section of the reactor building is shown in Fig. 1.

The containment consists of a cylindrical and a dome part. Connection between the cylindrical and dome parts is made with the help of a rigid ring beam in which the anchoring blocks of pre-stressing cables are placed. The wall thickness of cylindrical shell is 1.2m, the dome wall thickness is 1.1m. The containment structure is placed on the reinforced concrete slab at a level of +13.20m, thickness being 2.4m. This slab contains also supporting blocks of pre-stressing cables of the cylinder which are built-in there. The containment is made of concrete, grade B40 according to the Czech standards (CSN). The tightness of the containment is ensured by the steel liner of a thickness of 8 mm made of carbon steel.

A chart of the pre-stressing method is shown in Fig. 2. The cylindrical part of the containment is pre-stressed by 96 cables running in helical direction. The cable anchors are installed in the upper part of the ring beam, the bending of the cables takes place in the slab at a level of +13.20m. The dome part of the containment is pre-stressed by an orthogonal grid plan of pre-stressing cables formed by 36 cables. Always two cables are conducted against each other, anchors of one cable and bending of the other one are situated on one side. The anchoring blocks are installed from the ring beam side. The cables of the cylinder and dome parts are of the same structure and cross section. Cable preservation was made with grease during production, preservation of anchors was made after pre-stressing. The anchors are protected from climatic effects by means of the sheet covers installed.

The pre-stressing unbonded cables are conducted in polyethylene tubes. Every cable is formed by 450 wires featuring a diameter of 5 mm. Low-relaxation wire was used for production, its yield point being 1620 MPa. The initial pre-stressing force according to the design is 10 MN. On the basis of experience acquired during the pre-stressing of the 1st unit the anchor details were modified. These modifications have ensured better arrangement of wires on the anchor, and thus also a more even distribution of the pre-stressing force into the individual wires.

The containment function was verified during the structure integrity test (SIT). The test carried out on both the 1st and 2nd units was implemented as a combined test. The function was tested for strength at an overpressure of 460 kPa, the function was tested for tightness at an overpressure of 400 kPa. The SIT of the 1st unit was carried out in 1998, the SIT of the 2nd unit was carried out in 2000.

Overview of inspection activities during operation

For the purpose of ensuring full performance of the containment for the entire period of operation of the unit there was created an inspection programme of the containment structure. The inspection of the containment structure consists of the assessment of measurement of sensors of the permanently installed measurement systems and of the inspection of the conditions of concrete, liner and pre-stressing system. According to the frequency of the work carried out the inspections divide the work into two phases. During the first phase (the first 4 years of operation) the activities are carried out in full extent. Within the framework of the following phase a part of activities is carried out on an annual basis, and another part once in four years.

The inspection of the containment concrete consists of the following activities:

- inspection of the containment surface – to be carried out twice a year. Focused on checking for damage, corrosion of the reinforcement system and crack development.
- non-destructive concrete strength tests – carried out once a year, in the second phase once in four years.

Liner checks are carried out always when it is possible to enter into the containment. The inspection consists of the following activities:

- inspection of the coating for integrity and of the liner for damage.
- non-destructive measurement of liner thickness.
- check for tightness – carried out within the periodical test framework.

The checks of the pre-stressing system is carried out in the first phase of the inspection work once a year, in the second phase once in four years. The inspection consists of the following activities:

- inspection for humidity at the place of anchors and bends.
- inspection for integration of preservation at the place of anchors and bends and change in chemical properties of grease.
- Inspection of wires and anchors for damage.
- checks of the pre-stressing force by lift up tests.

The above listed activities are completed with the inspection of the cable removed. The dismantling of the cable will be made three times during operation, there will always be removed two cables of the cylinder and one cable of the dome. The inspection is focused on the condition of preservation, level of corrosion and damage to individual wires and mechanical properties of wires are verified on selected samples.

A new methodology has been developed for the lift up test. This methodology makes it possible to specify exactly the pre-stressing forces in the anchor. during the test the pressure in the press is recorded, as well as the force measured by Hottinger sensors and anchor lift from the supporting block (measured by the displacement sensors temporarily installed). This monitoring enables exact specification of the moment of anchor relief and thereby also the exact determination of the pre-stressing force in the cable.

Assessment measurement

In order to enable the inspection of the level of the containment pre-stressing, measurement systems are installed permanently on the structure, and these systems measure structure deformations and pre-stressing force in the cables. The measurement is carried out once a month, once a year the setting of the reactor building is measured. Inspection of the values measured is carried out at each measurement, a complete assessment is made once a year.

The following measurement systems are installed on the containment:

- NDS and SDM systems – these two systems consist of vibrating wire fitted during concrete pouring into the containment walls. The sensors are of four types and measure concrete deformation, temperature and horizontal shift in the middle of the height of the cylindrical part of the containment. The containment includes more than 240 sensors which are installed in it (246 on the 1st unit and 256 on the 2nd block).
- Hottinger system – this system is formed by strain-wire gauges stuck on the anchors of all cables of the cylinder and of the dome, i.e. 264 anchors measured. The sensors measure force in the anchor of the pre-stressing cables.
- MEM system – the system is formed by the sensors installed on the conduits of the pre-stressing cables. These sensors measure force in the cables by means of the magneto-elastic method. The sensors are placed on two cables of the cylinder and of the dome. The sensors on the dome cables are placed under the anchor and the cable bend. On cylinder cables they are placed under the anchor and at the middle of the cylinder height, on the 1st unit the sensors are installed in the lower part of the cylinder as well.
- HYNI system – the system measures the settlement of the reactor building by means of hydrostatic level control. Measurement is carried out with regard to the criterion of the reactor inclination. Since the reactor building is founded on a rocky bedding, the settlement was at a minimum level and there are virtually no changes anymore.

The distribution of sensors on the containment structure is illustrated in Fig. 3. At present it is the concrete creep that has the largest impact on the change in the state of stress and containment deformation. With regard to the age of the structure, the effects of the pre-stressing cable creep and of concrete shrinkage is minimum.

As an example of the values measured by vibrating wire of the NDS system, Fig. 4 illustrates the development of proportional deformations in the central part of the dome of the containment of the 2nd unit. From the graph it is possible to see the deformation in course of pre-stressing, and subsequently deformation due to concrete creep. The values measured also reflect the effects on the values measured in the case changes in outdoor temperatures during the year. If measurement is made in a period of several hours, the effects of outdoor temperature oscillation during the day has similar features. For the purpose of comparison, Fig. 5 shows the time history of temperatures in the dome during the same period.

The example of the pre-stressing force course on the cable is shown in Fig. 6. The graph states the values of pre-stressing force measured with the MEM system sensors on the cylinder of the 1st unit, cable no. 21a. From the graph it is possible to see the decrease in the pre-stressing force along the cable length due to friction, as well as decrease in the pre-stressing force over time as a result of concrete creep.

The comparison of measurement Hottinger, MEM and lift up tests is in Fig. 7 and 8. The graphs provide courses of regression of the average value of the pre-stressing force specified by the systems Hottinger and MEM (for sensors under the cable anchor) on the cylinder and on the dome of the 2nd unit. The graphs also state average values of pre-stressing force determined by the lift up test after 2000 hours from the pre-stressing and before the SIT.

Conclusion

The methodology of the in-service containment inspection presented in this paper enables us to inspect and monitor the containment of the 1st and 2nd units of the Temelin NPP, and provides a guarantee that the containment structure is able to perform the function of the last safety barrier.

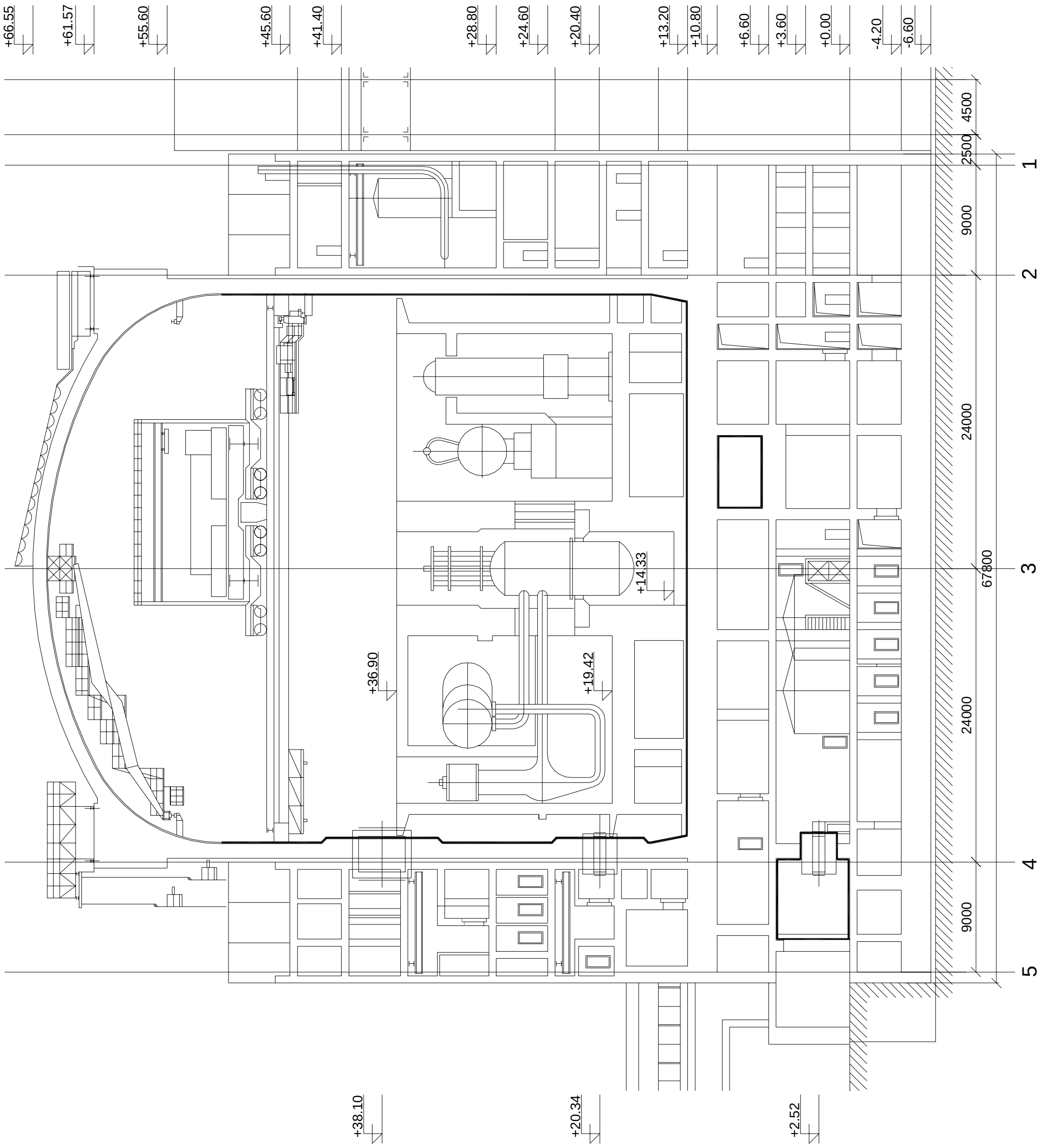

Fig. 1 Cross section of the reactor building

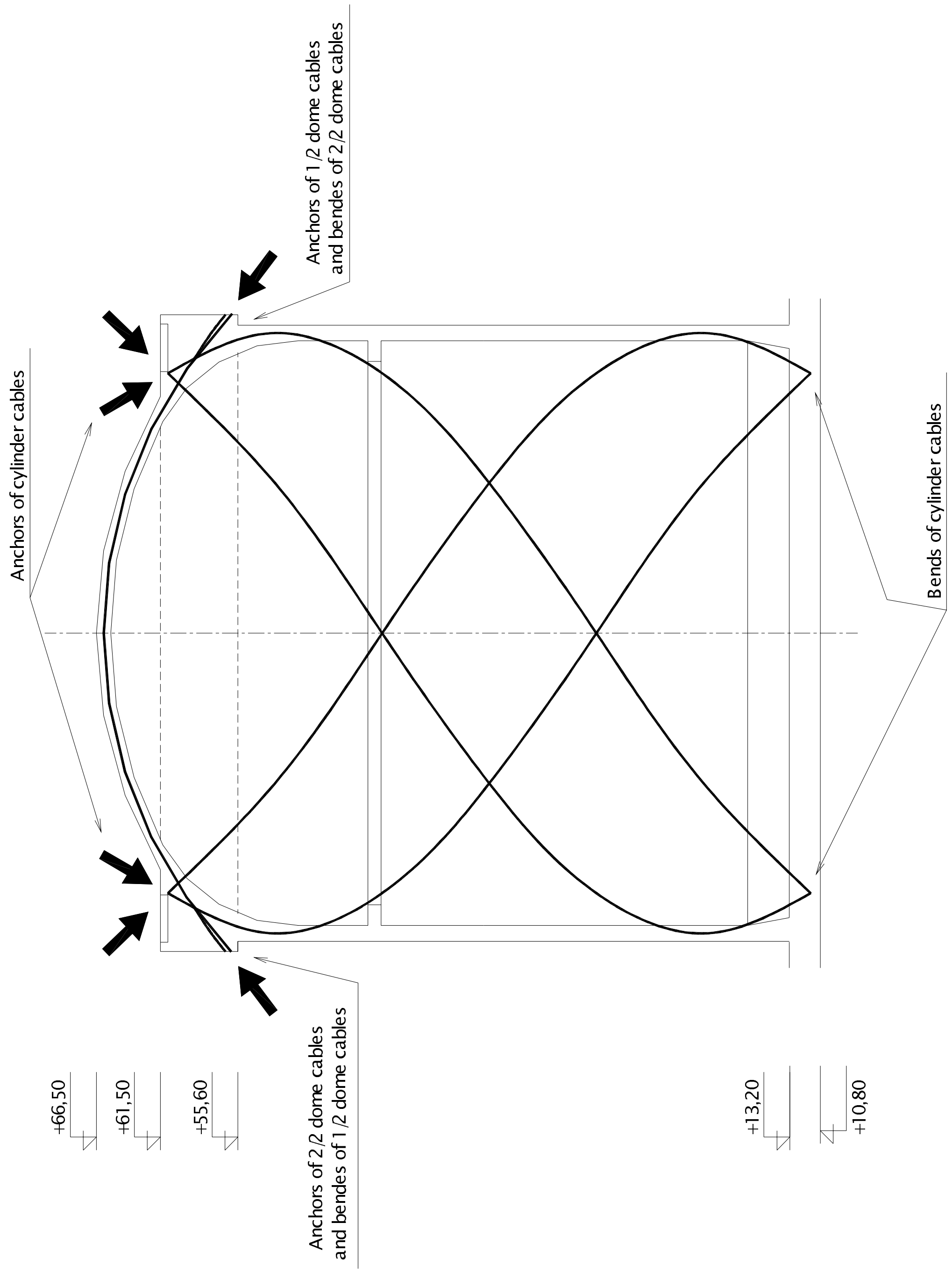

Fig. 2 Scheme of pre-stressing of containment

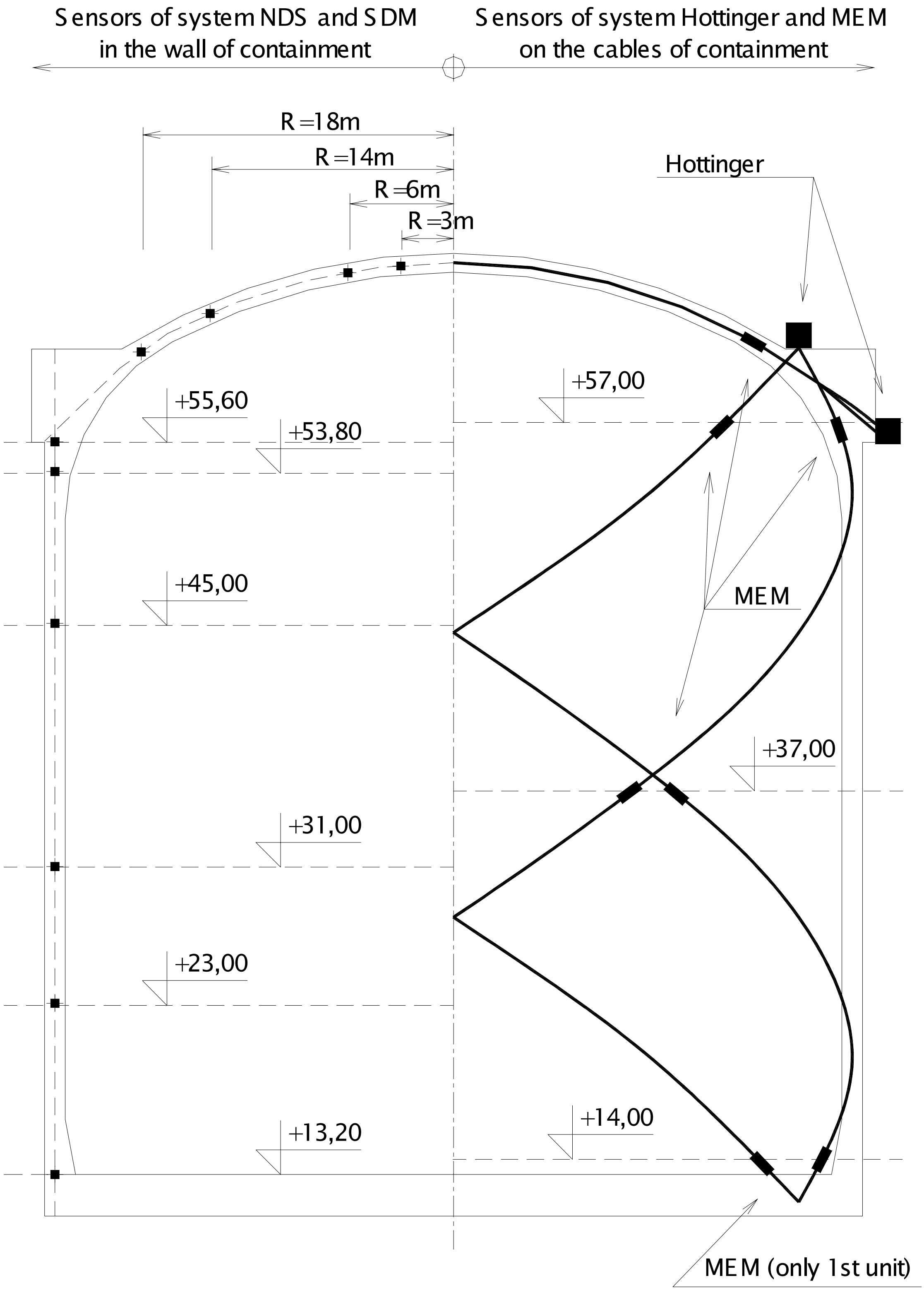

Fig. 3 Distribution of sensors on the containment structure

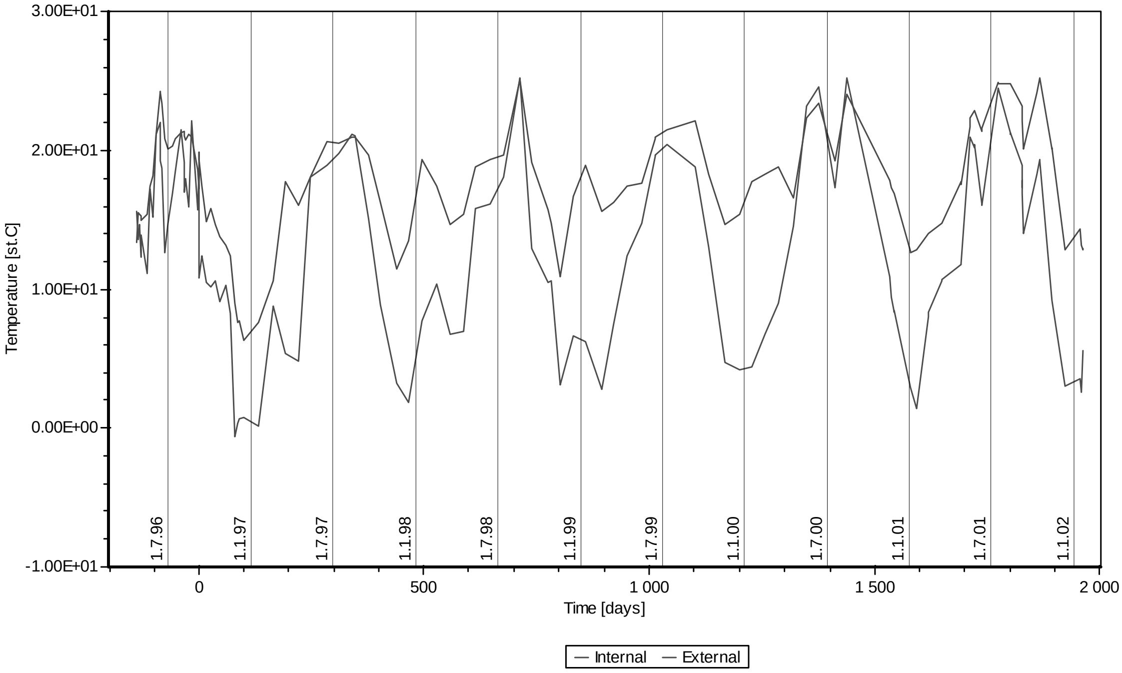

Fig. 4 Proportional deformations in the central part of the dome of the containment of the 2nd unit measured by string strain gauges of the NDS system, sensor type PSAS.

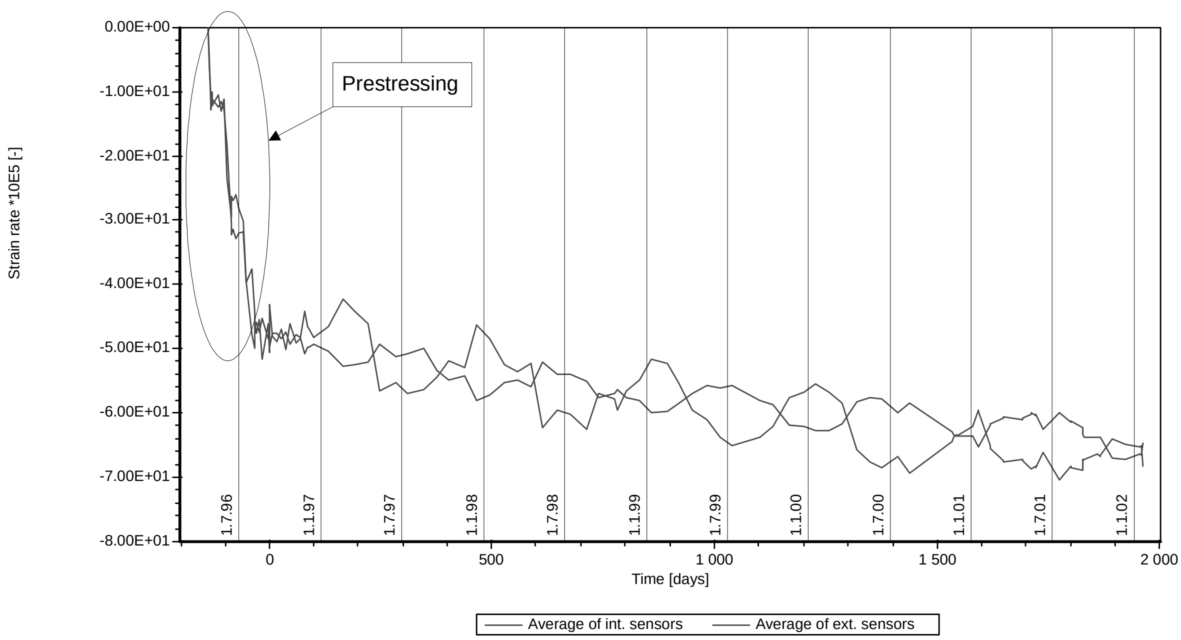

Fig. 5 The time history of temperatures in the dome during the same period as in Fig. 4.

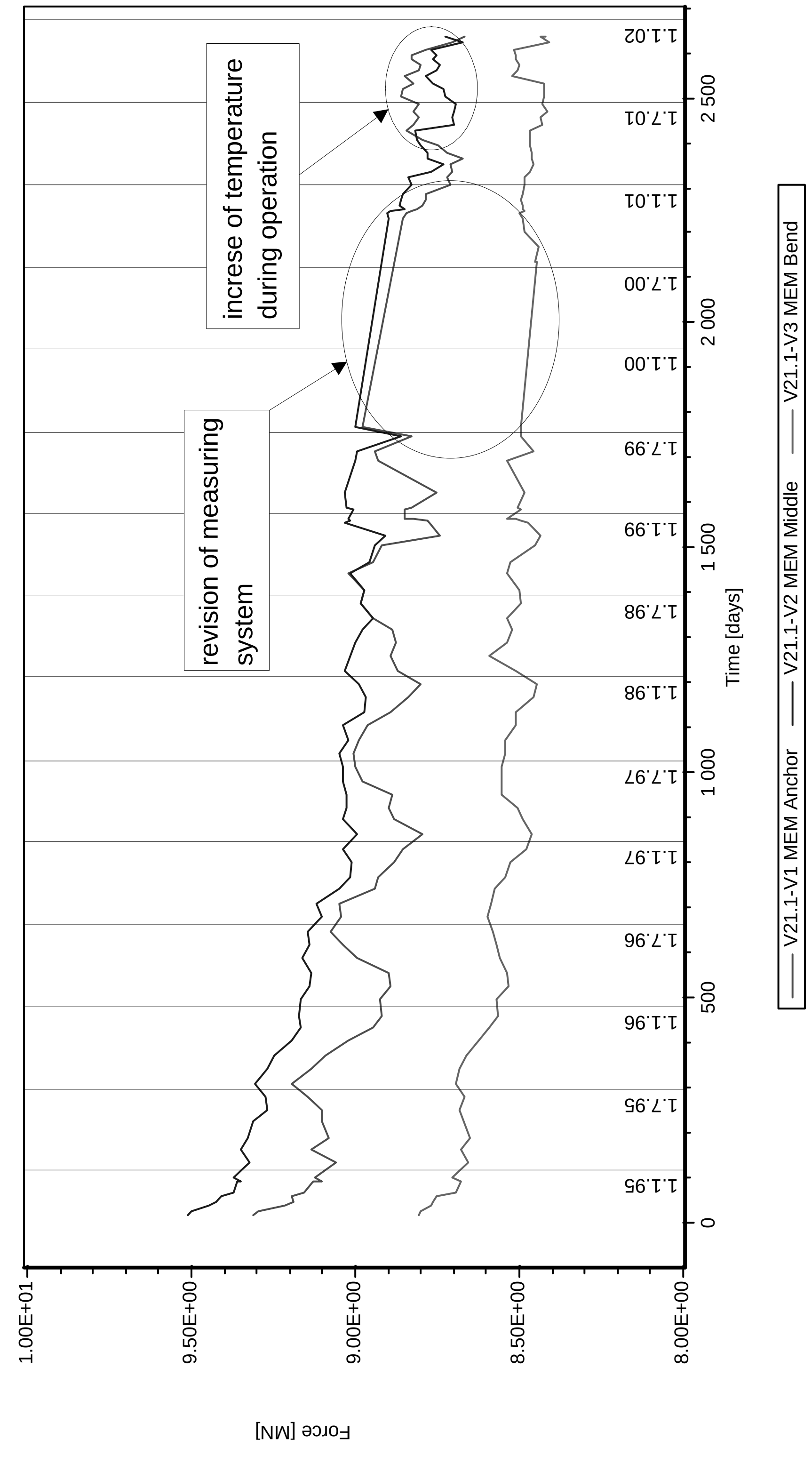

Fig. 6 The example of the pre-stressing force measured with the MEM system sensors - the cylinder of the 1st unit, cable no. 21a

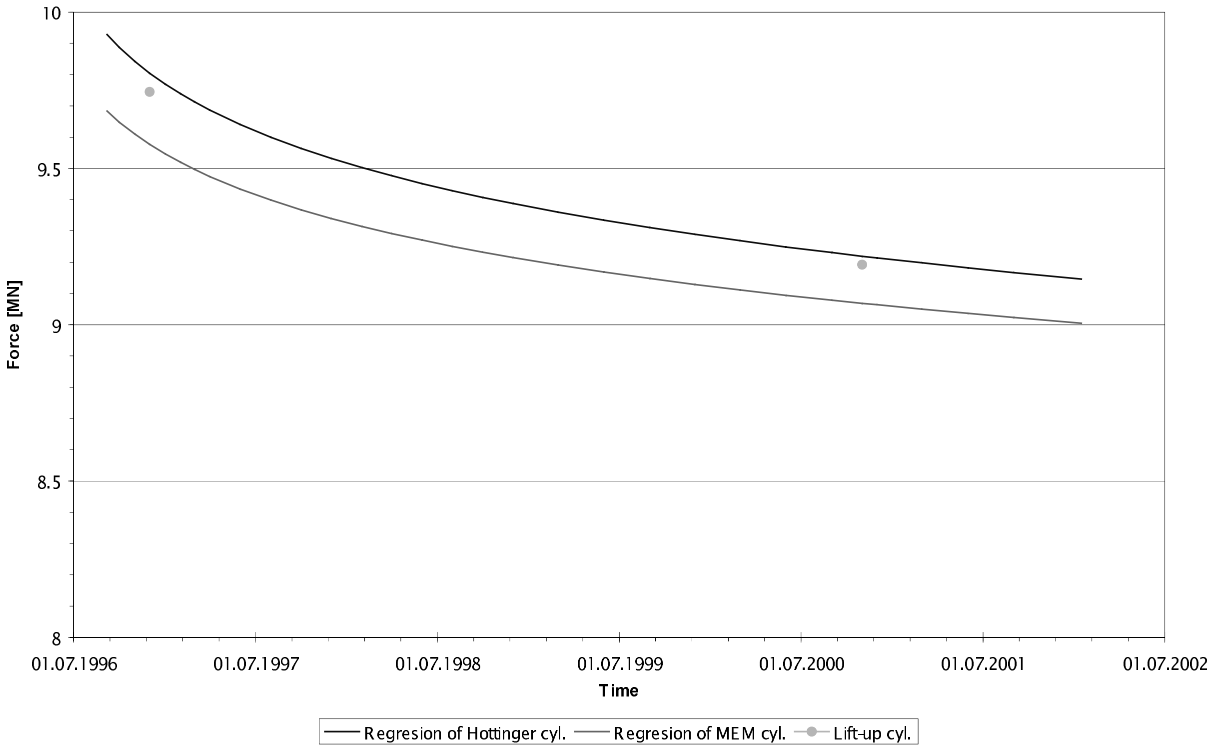

Fig. 7 The comparison of measurement Hottinger, MEM and lift up tests - the cylinder of the 2nd unit

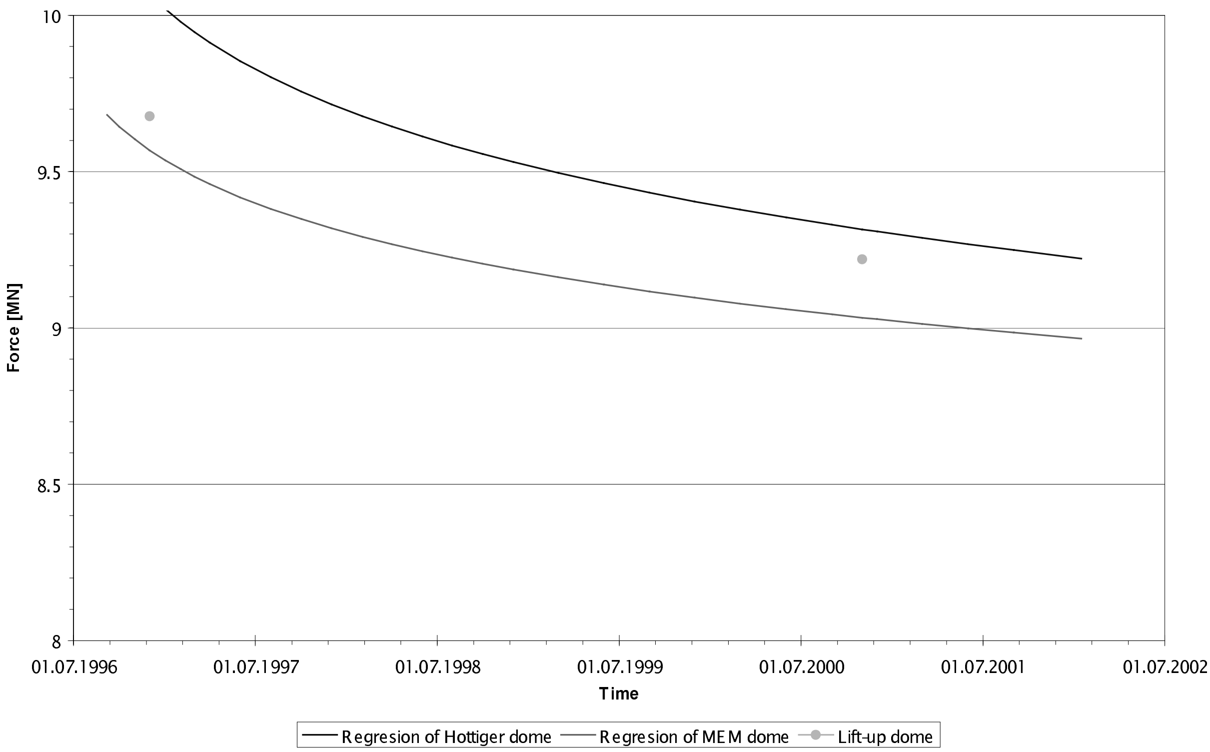

Fig. 8 The comparison of measurement Hottinger, MEM and lift up tests - the dome of the 2nd unit

REPAIR CRITERIA & METHODS OF REPAIR FOR CONCRETE STRUCTURES OF NUCLEAR POWER PLANTS

PARTICULAR APPLICATION ON NATURAL DRAUGHT COOLING TOWERS IN BELGIUM

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Abstract

A previous paper was presented at the OCDE Workshop held 22 – 23 March 2000 in Brussels explaining different aspects of the techniques used for “Instrumentation and monitoring of natural draught cooling towers in Belgium”.

These monitoring and preventive techniques are now applied in Belgium since already more then 10 years by Tractebel on the towers of the nuclear plants.

These huge constructions have to sustain considerable physical, chemical and biological loads. As one can figure out, and as years go by, these inspections showed deterioration of which type, progress, quantity eventually led to the need of repairing these structures.

The present paper goes over 4 main different sorts of defects (beam supports breaking, fast carbonation rate, concrete porosity, and a series of local deteriorations like insufficient concrete cover, cracking, gravel pockets, corroded reinforcement) encountered on 3 cooling towers situated in Belgium, and affecting the shell as well as the inner structures.

The diagnosis, the choice of the appropriate repair techniques and products which will avoid having to face much higher costs in the future are explained.

It also gives an illustration of the works carried on site and points out the uncommon and complex aspects the treatment of such a construction implies (planning, both horizontal and vertical curved shape, works at great height, …).

1. Introduction

A cooling tower, whether at a fossil-fired or a nuclear power plant, where the circulation water is cooled after having left the condenser, is a main component as it provides the cold source in the thermodynamic cycle of the turbine.

On account of the water flow rate to be treated and the volume of air required, the cooling towers, built of reinforced concrete, are structures of hyperboloid revolution that are very impressive by their size and shape (up to 160 m high for the large installations). The developed surface area of the thin concrete shell of these towers may reach several tens of thousands of square metres. By their nature, the cooling towers have to sustain considerable physical, chemical and biological loads.

The inner structure is a construction of beams and columns designed in order to provide the support of the air/water exchange material allowing this construction to perform its cooling function.

While for a long time concrete constructions were considered capable of defying the years without problems, this simplistic approach has since been abandoned, as it is now recognised that, like other materials, concrete is affected by ageing and by various illnesses that need to be treated and kept under control if the concrete is to reach its optimal operational life. These constructions were generally ordered by the operators as a turnkey component and, as such, were not verified (calculations and works) by the engineering consultants as Tractebel.

Moreover, in the seventies, several cooling towers even collapsed, like in Ferrybridge (England) or Bouchain (France) reminding all structural and electrical engineers that a major failure of this essential power plant component was always possible.

2. Technical approach

2.1 General description and diagnosis

The concrete of a cooling tower, exposed to the aggressions of its environment and to the operating conditions, presents also a number of characteristics that are specific to it as compared to other structures. Therefore, investigations are first required to identify the causes of the deteriorations, as the causes will dictate the methods and materials required for a successful restoration.

The origins of the deteriorations may be chemical, physical or biological: wind action, the structure's own gravity, inside/outside temperature gradient, insulation differential, rain action, air pollution, soil-structure interaction, water vapour and proliferation of algae or mosses on the shell leading to cracking, deficiencies inherent to the concrete such as the presence of gravel nests, chipping and spalling, deteriorations caused by carbonation, alkali-silica reaction or by chlorides.

In the case of a cooling tower, all these deteriorations make fragile a structure which is constantly surrounded by an atmosphere saturated with water vapour and which is exposed to thermal differential stresses the major part of its existence.

These environmental conditions lead to the slow deteriorations of the tower's components: reinforcement corrosion, concrete leaching.

2.2 Treatment guidelines

Accordingly, treating a component such as the shell of a cooling tower requires many investigations, core sampling at significant locations, laboratory analyses, selection and validation of particular treatment processes and procedures that are suitable, considering also the access constraints and the size of the surfaces needing treatment. In the case of serious degradation of the concrete performances, a new calculation of the shell might need to be performed that could lead to strengthening measures being prescribed.

In general and if the chemical deteriorations are limited, treatment involves repair of the physical deteriorations, followed by preventive treatment of the concrete against aggressive elements.

Repair of the physical deteriorations comprises the treatment of the reinforcement and the replacement of the damaged concrete so as to return the tower as close as possible to its initial condition.

This first action is then followed by surface treatment. Taking into account the geometrical characteristics and the deformation of the shell, this treatment usually involves applying to the inner face, which is the one exposed to the water vapour, a treatment more impervious than the one applied to the outer face. As a result, the inner face is sealed off against ingress of water into the concrete, and the concrete is allowed to "breathe" on the outside. Also, as the two faces are treated, penetration of carbon gases and other aggressive elements is prevented and chemical reactions can be stopped.

In practice the review of the way the treatment is to be implemented is completed by a comparative appraisal of the methods of accessing any point of the inside and outside of the shell. Complex problems are taken into consideration in this review, relating to the height of the structure, its both horizontal and vertical curved shape and the necessity of supplying to any point the liquids and products under required pressure for the treatment. For instance, high pressure water-jet cleaning requires water at 400 bar (or even more) to be provided at great height. Also, all the means of access have to present sufficient safety regarding operators and products, while making possible the rapid progress and quality of the work.

Finally, as the works are only performed during a stop of the tower, usually meaning lowering or stopping the production, these equipments are to be suited (in number, speed and ease of use) with the shortest possible working schedule.

2.3 Encountered problems

The 3 first examples concern degradation affecting the shell of 3 power plant cooling towers (Ruien 5/6, Doel 3 and Tihange 3).

The last example concerns problems affecting the inner structure of the Tihange 3 cooling tower.

3. Ruien power plant units 5/6

3.1 Main features

Cross-flow type cooling tower built in the years 1972-1973 :

- diameter at the ground level (cold water basin not included)	: 60.90 m
- diameter at the lintel	: 54.30 m
- diameter at the throat	: 41.60 m
- diameter at the top	: 48.50 m
- height at the lintel	: 16.15 m
- height at the throat	: 62.58 m
- total height	: 94.65 m
- shell thickness at the lintel (varying up to 28 m)	: 0.70 m
- shell thickness from elev.28 m to the top	: 0.12 m

The reinforcement is made of only one centred reinforcement layer.

As significant vertical cracks appeared after a few years, the builder placed at his own costs under the 10-year guarantee, an additional external reinforcement to avoid a potential collapse of the structure. This involved placing 42 cables T13 (each tensioned with 30 kN) every 0.80 m from the throat to the top.

3.2 Problem description - diagnosis

The cooling tower of the Ruien power plant was found to be in the following condition:

- shell: cracking (partly passing through), local deteriorations, gravel nests (some going right through the shell), corroded reinforcement, uneven concrete surface, though fairly good concrete quality in the sound parts of the shell, carbonation depth < 15 mm (pict. 1).

- shell supports: cracking and deteriorations under chemical attack.

The nature, the quantity and the size of the shell cracks were such that the initially monolithic structure had now to be considered as an articulated one.
Also, the concrete needed treatment to stop leaching and protect the reinforcement.

3.3. Treatment definition

In a first phase, core samples were taken from the shell in order to identify the physical and chemical properties of the concrete, so that intended treatments could be assessed and their chances of success evaluated.

After this, the phases of the treatment were defined:

- complete cleaning of the inner and outer surfaces (pressure jetting with water and/or sandblasting) (pict. 2);

- removal of all the portions of poor quality concrete (in and out);

- treatment of reinforcement (in and out) : cleaning (all around the bar), replacement when necessary and protection against corrosion with an hydraulic-based product;

- replacement of the removed concrete with a new hydraulic cement-based compound, filling of the grid nests and correcting of the unevenness with the same product (in and out);
- complete shaping (inside only) with a fine polymer improved hydraulic rendering cement (6 kg/m2);
- inner face protection: application of impervious coats that would remain flexible enough to follow structural deformation. On an impregnation epoxy resin coat, a polyurethane-based multi-layer flexible system (2 mm) was laid, protected afterwards with a final coat (40 µm) providing the UV radiation protection;
- outer face protection: application of protective coats witch allow the concrete to breathe and which are not sensitive to structural deformation. A hydraulic elastic polymer mortar was applied in two phases of 1,5 kg/m2 and 3 kg/m2, the first layer applied with a trowel, the second gun-applied.

These coats provide the concrete with a barrier against attack by various substances. Moreover, the application of cement-based products brings a complementary quantity of "fresh" lime that contributes to stabilize and partly restore the alkaline properties of the existing concrete, improving thus the protection of the reinforcement and restoring the intrinsic strength of the concrete.

In a second phase, tests were performed on selected areas in order to determine the corrosion level of the reinforcement and validate the product application methods that proved the most effective.

3.4. Works

A particular problem was faced with this cooling tower due to the pre-cast cables being located outside the upper half of the concrete shell. This had to be taken into account when carrying out the various operations (movement of hung scaffolding, pressure cleaning, application of the coating products) (pict. 3).

Concerning the shell supports, the bases of the columns were injected with a sizing epoxy-based resin. However the small opening of the cracks of the columns did not make their injection possible. The columns have been treated with the same product as the inner face, however in two layers only.

The works have been carried out under cover of a quality assurance program and controlled by the experts of an insurance company allowing it to be covered by a 10-year full warranty conditional to regular inspections.

Representing the treatment of 12,000 m^2 per face plus the supporting columns, they have been carried out within a 3 months period (of which only one month of real working days due to bad weather conditions) for the inner face and about one year for the outer face (with an average working ratio of 45%, winter stop included).

4. DOEL 3 Power plant

4.1. Main features

Counter-flow type cooling tower build in the years 1981-1983 :

- diameter at the ground level (cold water basin not included)	: 141.70 m
- diameter at the lintel	: 133.88 m
- diameter at the throat	: 76.64 m
- diameter at the top	: 83.56 m

- height at the lintel	: 12.70 m
- height at the throat	: 108.70 m
- total height	: 167.28 m
- shell thickness at the lintel (varying along the first 10 m)	: 0.85 to 0.25 m
- shell thickness from elev. 10 m to 104 m	: 0.25 to 0.18 m
- shell thickness from elev. 104 m to the top	: 0.18 m

The reinforcement is made of two reinforcement layers, the theoretical concrete cover is larger than 40 mm.

4.2. Problem description - diagnosis

The cooling tower of Doel 3 is affected by an illness that results of its shell outer face incurring a too fast carbonation rate (penetration of carbon dioxide gas in the concrete). This phenomenon, which was demonstrated by a series of tests carried out in Belgian laboratories, leads to a modification of the alkalinity of the concrete that protects the reinforcement. With time the reinforcements get corroded and cause serious deteriorations as the concrete bursts. In our case, the penetration depth was about 30 to 35 mm, which is almost equal the concrete cover of the reinforcement.

4.3. Treatment definition

The propagation of carbonation can be arrested by the application of a coat that prevents the carbon dioxide gas penetrating into the concrete.
The suitable products are selected following in situ tests and manufacturer's specifications. The treatment of the outer face of the wall is however not sufficient. Further investigations resulted in an impervious coating being planned for the inner surface so as to avoid ingress into and diffusion within the concrete of water vapour, since this would have induced flaking of the paint on the outer side. As the concrete was not much cracked, there was no need for flexible coating. An impervious epoxy-based system was chosen for the inner protection, it is applied in 2 layers (non pigmented impregnation layer of about 500 g/m2 and pigmented final coat of about 300 g/m2).

The outer protection consists of 3 layers of a one component synthetic (plastic mixture based on PVC copolymers) paint (300, 400 and 300 g/m2 of different shades).

The latter effectively protects the concrete by preventing carbon dioxide penetration (CO_2 diffusion coefficient μ = 3.05 $10^{6)}$ whilst allowing the concrete to breathe (permeable to air and water vapour) and offering a satisfactory resistance against ageing (chiefly UV radiation). Because of the unpredictable weather in our countries, the paint was chosen (solvent-based) so that it could be applied on wet surfaces and drying time would be short (1 to 2 hours).

4.4. Works

Most of the treatment operations are similar to those already described, though a number of particularities exist at Doel 3.

For instance, as the outer face of the shell had its concrete roughened in alternance by two types of vertical ribs (the one long and slender and the other short and broad), such surface offering better behaviour under wind loads. After a few years, the corrosion of the longitudinal reinforcement that had been placed in the slender ribs (which had a very thin (5 to 15 mm) concrete cover) induced a general cracking of these ribs (pict. 5). A new calculation of the shell showed that the slender ribs could be completely removed from the tower. This was in fact less expensive and more feasible than trying to repair them. These works were carried out 2 years before the treatment of the complete shell was considered. A new problem occurred because of the remains of the reinforcement that previously linked the ribs to the shell. This steel had been cut close to the shell and was corroded due to an insufficient protection. These countless rust points required a particular treatment method that was defined after detailed trials and tests (pict. 6).

Another particularity resides in the fact that the plant of Doel (4 units) is only provided with 2 cooling towers, each one serving 2 units (D1 coupled with D4 and D2 with D3). The stop of two units is thus required if work is to be performed inside a tower unless the thermal conditions of the outfall allows the circulation water of one unit to bypass the tower while the other is stopped. In our case, the entire treatment inside the cooling tower had to be carried out within the strictly limited period that unit 3 was shut down for steam generator replacement. In this short period had to be covered the assembly/disassembly of the fixed and mobile scaffolding installations, the automatic remote-controlled and manual cleaning devices, and of course all the cleaning, repair and painting operations. Furthermore, the presence of 2 stiffening rings on the inner face of the tower (each protruding by some 1.25 m) did not make things easier (pict. 4). With some precautions, the painting of the inside was done using an airless spray gun. Thanks to good forward planning, the operation was completed successfully within the six weeks time allocated.

On the contrary, the restoration work on the outer face of the shell took much longer then expected, as the necessity appeared for corrective treatment of the ribs. Dealing also with the weather and temperature conditions (leading to an average working ratio of 45%, winter stop included), the work could not be completed in less than one and a half years despite the simultaneous use of 4 cradles. In order to obtain an homogeneous final shade, the last coat (representing 48,000 m^2 and manufactured in one batch) was nevertheless applied in only one week.

The whole work was also controlled by the experts of an insurance company allowing it to be covered by a 10-year full warranty conditional to regular inspections. The first of these inspections took place after one year and did not reveal any visible deterioration.

5. Tihange 3 Power plant

5.1. Main features

Cross-flow type cooling tower built in the years 1981 - 1983 :

- diameter at the ground level (cold water basin not included)	: 120,5 m
- diameter at the lintel	: 111,30 m
- diameter at the throat	: 62,9 m
- diameter at the crown	: 67,8 m
- height at the lintel	: 8,7 m
- height at the throat	: 109,3 m
- total height	: 157,5 m
- shell thickness at the lintel (varying along the first 10 m)	: 0,86 to 0,21 m
- shell thickness from level 10 m to 100 m	: 0,21 m
- shell thickness from level 100 m to 115 m	: 0,21 to 0,17 m
- shell thickness from level 115 m to 140 m	: 0,17 m
- shell thickness from level 140 m to the top	: 0,17 to 0,50 m

The steel reinforcement is made of two layers, the theoretical concrete cover is larger than or equal to 40 mm.

5.2. Damage description - diagnosis

Since a few years, the concrete of the external side of the shell of Tihange 3 cooling tower became ochre-coloured. The outer side of the shell had become ochre several years already below the throat, due to this part of the face being exposed to rainfall. More recently this was increased by percolation which formed a permanent wet patch presenting traces of calcite, several hundreds of m² covering the first 15 casting rings (each of 1.5 m high).

In addition to the inspection routinely performed in the scope of the monitoring program, several particular analysis were carried out to focus on the phenomenon which induced this particular pathology. These consisted of :

- a chemical, petrography and microscopic analysis of core samples taken within and outside the wet patches;
- a chemical analysis (particular for Fe content) and microscopic examination of algae fragments taken from the internal face of the shell, and for comparison purposes from another cooling tower at the same site;
- a corrosion study of the reinforcement by electrical potential measurements in the wet patches and in a totally opposite area;

The overall analysis of these two sets of investigations permitted drawing coherent conclusions which can be outlined as follows :

- the absence of correlation between the deformation and the cracking hints at the concrete itself being at fault;
- the reinforcement corrosion was found low except in the wet patches where it can be said medium to strong;
- the concrete porosity is very high, its density and compressive strength are low;
- the ochre colouring does not reveal reinforcement corrosion, but appears to result from a mineralogical transformation of ferrous particles present in the cement, resulting of a drop in pH due to concrete carbonation. The carbonation front matches exactly the coloured zone.

The percolation observed in the wet area in fact results from the too high porosity of the concrete. The permeability throughout the mass of the concrete induces the percolation, the engine of this being the pressure gradient between the inside and the outside of the tower..

5.3. Determination of the treatment

Water percolation jeopardizes the durability of the tower as it, in the longer time, results in reinforcement corrosion and a washing away of the concrete mix constituents.

We therefore found it necessary to fight percolation by impeding the water to penetrate into the concrete, by applying an inner waterproofing coating composed of two layers of epoxy, in addition to the primer. This treatment is adequate because the cracking is low. Several other reasons (limited time, importance of permanent cost, technical risks, …) resulted in the decision being taken to treat all the entire inner surface rather than just the affected area.

No treatment was advised for the outer face as it is important to be able to ascertain the efficiency of the inner coating. The outer face has to be left as is not to mask a possible evolution. Also no technical arguments (since carbonation and corrosion are low) can justify outer treatment which as we know is very costly, and moreover, it allows the concrete to "breathe".

5.4. Works

The work was quite similar to that done on the inside of the Doel 3 cooling tower shell, applying the same methods and essentially requiring a sufficiently long period of stoppage of the plant (pict. 7).

A steam generator replacement which was planned at Ti3 in the summer '98 offered that suitable window to treat the cooling tower. However due to other maintenance work to the tower, only 6 weeks were available to achieve this treatment.

The challenge was met : the first site meeting was held on 10 June and the last lick of paint was applied on 20 July, despite 12 days in all being lost due to bad weather.

The attached tables 1 and 2 illustrate the progress of the repair and coating of the 37.600 m² of the inner face of the structure, which involved applying some 37T of product. Appendix 3 gives an idea of the poor weather conditions (essentially rainy days, but wind can be embarrassing as wel), and their impact is reflected on the progress curve.

Like at Ruien and Doel the work was covered by Quality Assurance requirements and supervised by experts so as to insure the 10 year guarantee.

In addition to the practical aspects of this type of work which are similar to the already mentioned examples, one point was analyzed particularly : the structural stability of the crown walkway from which the painter's cradles had to be suspended. Depending on the orientation and the suspension points, these cradles induce tilting loads of several ton/m. Indeed this tower inspection walkway is not an integral part of the shell. It is composed of a series of prefabricated "U" elements, each fixed individually by means of 2 bolts on the top edge of the shell and 1 bent rebar at each end. These U elements are each 3.4 m long, are straight and are spaced 12 cm from each other, forming a polygon on top of the crown (pict. 8).

The checking of the stability of these boxes was favourable providing their anchoring, which were already 15 years old, had retained their original strength.

This was verified by :

- tapping and visual inspection of the seals (absence of cracking, rust, dull sound)
- core samples to test the quality of the material
- verify the compliance between the reinforced drawings and the execution, through a magnetic "X Ray" of the elements in order to check the position, the number, the diameter and the cover of the reinforcement.

The confirmation of the quality of the concrete and the suitability of the reinforcement compared to the calculation allowed to go ahead with the treatment with full knowledge of the facts.

6. TIHANGE 3

6.1. Introduction

Not only the shells of these towers can be source of problems !

As said before, the cooling tower is a heat exchanger the function of which is to cool down the water of the third circuit of a nuclear power plant, cooling itself the turbine condenser (circulation water).

This is obtained by a heat exchange between air (up-draught) and water. The cooling water is sprayed on the inner horizontal surface (representing about 10.000 m², i.e. about 3 football fields!) by a net of channels and pipes. It is sprinkled above an exchange material (called packing) across which it trickles (pict. 9). In normal conditions, the water flow rate represents an amount of water of 4 l/m²/s, in winter conditions this value can be doubled.

The water is collected at the foot of the tower in a basin and then flows by gravity to the return circuit, to a discharge in the river or to the pumping station depending on the thermal conditions.

On Thursday 3 January 2002, the break of 2 brackets of the supporting structure resulted in the collapse of 2 beams and the packing they were supporting (pict. 10). The flood entrained the debris which was stopped by the protection screens placed at the entry of the returning pipes. The inlet of the pipes became rapidly clogged and eventually the basin flowed over. Due to the geographical position of the tower, the overflow flooded to some of the NPP buildings.

6.2. Findings

The NPP operator ELECTRABEL asked TRACTEBEL Energy Engineering to analyse the accident. The next day some quick investigations were made (pict. 11):

- the break affects two brackets embedded at the lower face of principal beams that supported the secondary beams, themselves supporting the packing;
- the affected brackets are anchored at the lower face of the principal beams as if they were suspended;
- the reinforcement steel that became bare and the rupture surface do not show any trace of corrosion or dirt;
- the concrete of the rupture surface feels sound;
- the two fallen beams are situated in the same area, present the same constructive characteristics (geometry, supports, …), provide an identical function and incurred the same type of break.

6.3. Analysis

As the rupture surface was clean and there is no steel corrosion, one may consider that the failure was sudden (brittle break, like a lump of sugar) and corresponds to a shear break of the concrete. The concrete characteristics seemed good, core samples have nevertheless been taken in the wrecked brackets to verify this aspect. Results are satisfying.

Other causes were thus to be searched in an inadequate match between the design and the actual load applied to the bracket.

The consultants have investigated in the following directions while the cooling tower was stopped (in fact, the NPP can work with the tower by-pass, the circulation water is not cooled as it is sent to the basin before being sprayed in the sprinkling system) :

- loading condition of the structure
- resistance capacities of the structure
- visual inspection of the structure

a) Loading state

An eventual overload of the bracket can only result from an excess of weight applied on the supported beam, originating from the packing. This overweight of the packing may result from its clogging or winter service conditions (to minimize freezing problems, the flow is only distributed on the periphery of the packing, which means that for a same flow rate, the concerned wet surface receives a doubled flow, increasing the water weight as well).

Measurements have been carried out on samples of the packing near the wrecked area. A new problem arises because the measure is made on a dry packing as the cooling tower is stopped, and therefore the water quantity has to be estimated.

The packing itself is made of different elements (part of their weight doesn't change, while other parts of the packing can be fouled, increasing their weight). Considering a dead load of about 0.5 kN/m³, the surprise was great when it was discovered that some packing elements weighed more than 2 kN/m³. The problem was then to evaluate the correspondent weight of water : part of it trickling across the packing, part retained by the clog matter. A variation formula has been used to do this, considering that the quantity of water is a function of the degree of clogging, and that it can reach up to 1,5 times the dry weight of clog matter in the worst case.

b) Structural analysis

Concurrent with this analysis, a complete study of the structural elements was carried out, taking into account the initial design calculation codes, to determine the available safety margin for each element on basis of the as built drawings. This study has enabled us to point out which of the elements were theoretically the most at risk.

c) Visual inspection

Based on the above mentioned study, a visual inspection was performed on the "most at risk" elements. The discovery of unexpected damage called for a comprehensive visual inspection involving more than a thousand spots (pict. 12 & 13).

d) Load test of the wrecked beams

The two fallen beams have been examined and showed, over their entire length, regular cracking at the inferior face, going up on each side to mid-height. They were taken to a laboratory where they were tested to verify if the load they had supported had changed their future behaviour (still elastic or not).

6.4 Results and diagnose

6.4.1. Packing support structure (low level)

- The failure observed is due to an overload of the packing. This overload has broken 2 brackets of a series of 24 supporting the longest beams of the tower. Their visual inspection showed a generalized cracking.

- The tests performed on the 2 beams have shown that they were still in good shape (however they had fallen down!), breaking by flexion at their design load.

- The visual inspection of the rest of the structure highlighted the presence of degradations (mostly at the extremities of the beams and/or the brackets), some of them being severe enough to represent a danger for the structure.

6.4.2. Supports of the spraying system (upper level)

- The same kind of degradation was observed randomly distributed on 10% of the supports (pict. 14 & 15). As our theoretical study revealed, the thermal effects due to the temperature variations (stop/service conditions, day/night, sun/shadow, winter/summer, …) develop horizontal loads whose importance is close to the vertical loads. Originally nothing was planned to limit concrete/concrete friction between the beam and its support.

To summarize, two main causes were demonstrated :

- the clogging of the packing, leading to its overweight;
- an underestimation of the horizontal loads caused by a lack of support material.

6.5. Treatment and repair works

Twenty-two new steel brackets were designed to support both the initial one and its beam. They have been ordered, fabricated and placed within 3 weeks on the lower level to replace the brackets wrecked by overweight of the packing (pict. 16).

The brackets and beams whose extremities are severely cracked have been supported by scaffolding that will be left in place until the next stop.

At the upper level, the configuration of the brackets allowed us to design another steel system and reinforce beam and brackets that were cracked (pict. 17). These supports were designed, ordered, fabricated and placed within a week.

6.6. Conclusions

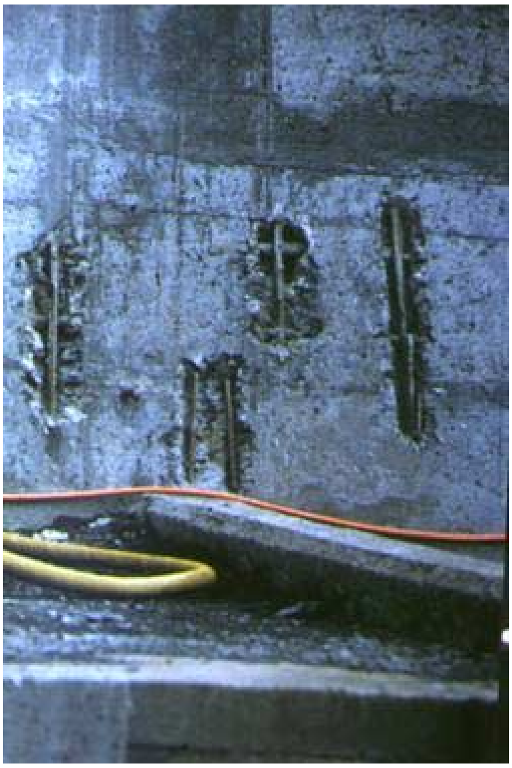
This last example shows once more, if necessary, the need of regular inspections and/or monitoring to avoid the risks of the occurrence of severe and costly damages, possibly even human consequences.

In this case, we proposed to the operator to monitor the packing weight by two means : continuous weighing of selected volumes of packing and by equipping the two new beams (replacing the wrecked beams) with strain gauges providing a continuous measure of the actual weight carried by the structure. The study that was made gave the opportunity to set the adequate levels of alarm.
The water level of the basin shall also be monitored to avoid another flood.

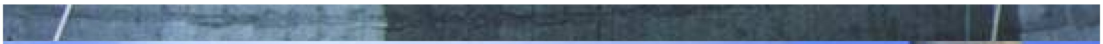
7. Appendices and pictures

7.1. Ruien ¾ : repair and painting works

Picture 1



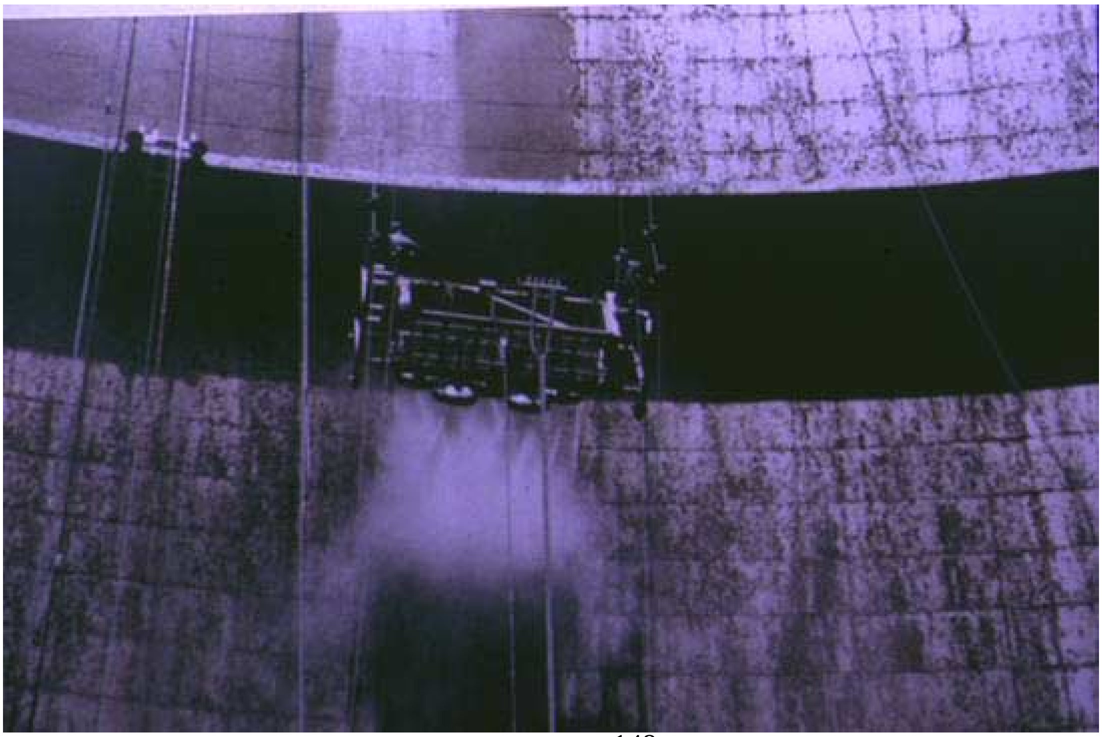
Picture 2



Picture 3

7.2. Doel 3 : painting works

Picture 4

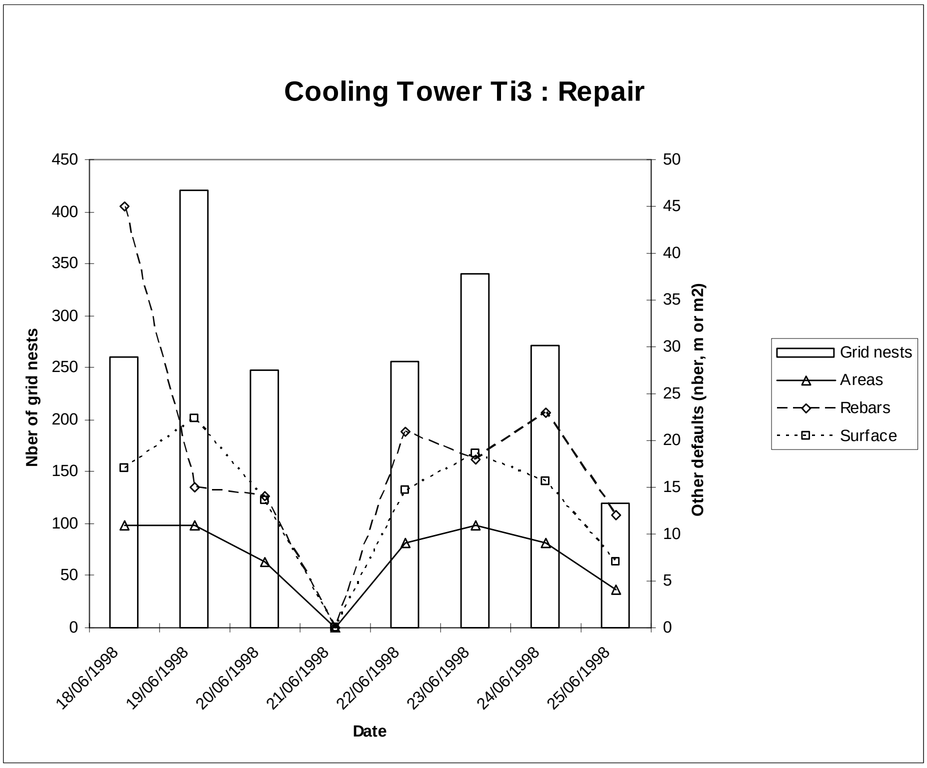


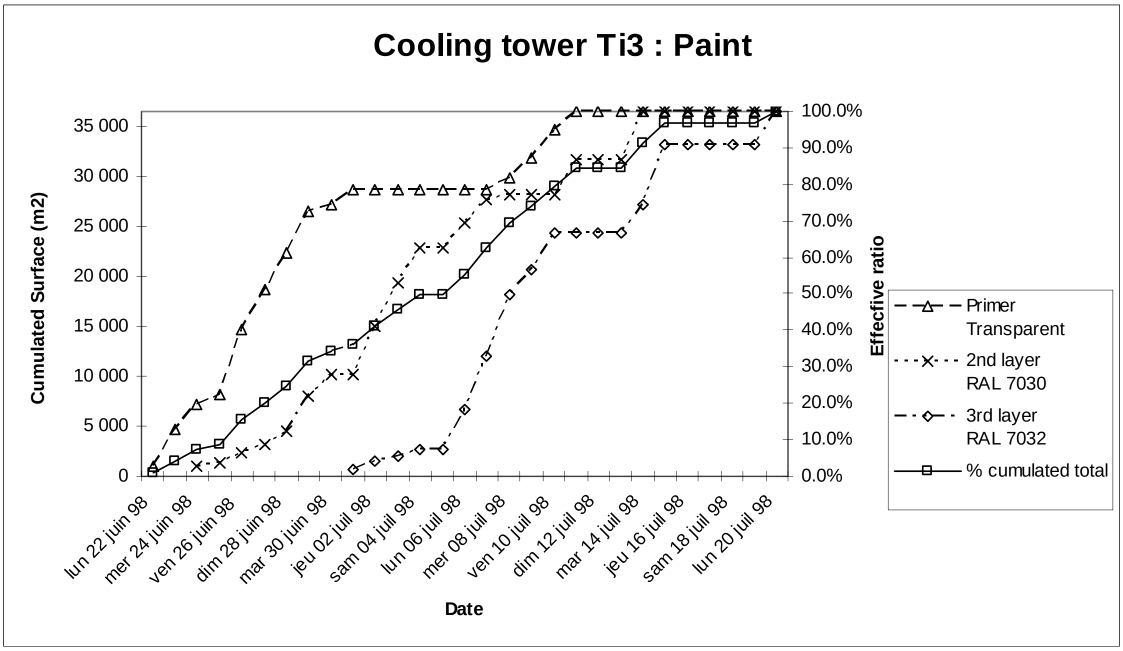
Picture 5

Picture 6

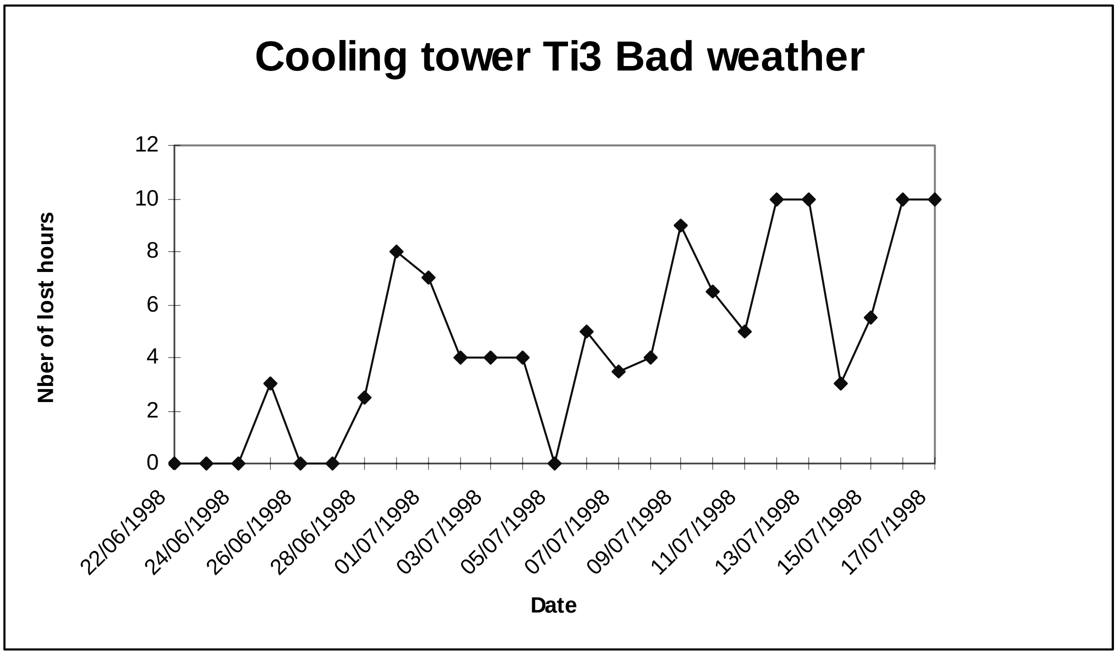
7.3. Tihange 3 : painting works

Appendix 1
Appendix 2



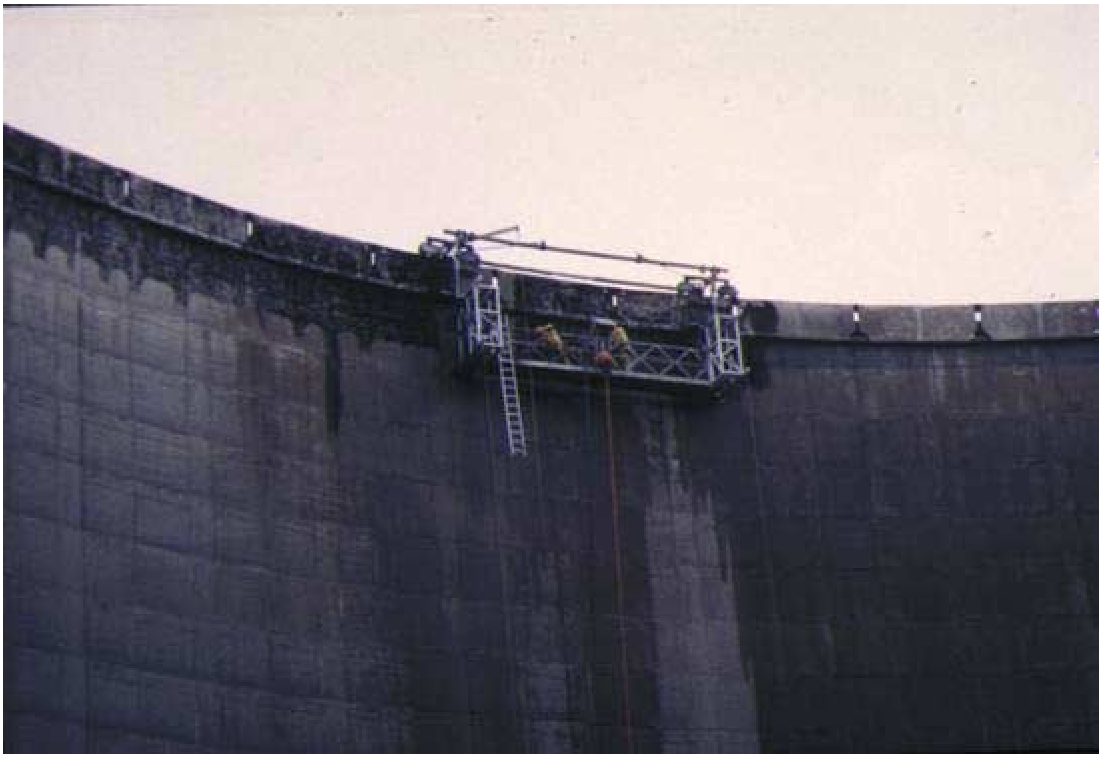


Appendix 3



Picture 7

Picture 8



7.4. Tihange 3 : brackets breaking

Picture 9

Picture 10

Picture 11

Picture 12

Picture 13

Picture 14

Picture 15

Picture 16

Picture 17

Post-Fire Damage Assessment Procedures for Nuclear Power Plant Structures

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Abstract

The examination, inspection, maintenance and testing of all plant and structures that may affect nuclear safety on Nuclear Power Plants is of prime importance. It is essential that all nuclear safety related structures must be maintained throughout their operational life in such a way that they are always fit for purpose and capable of meeting their nuclear safety role as required. In order to do this they must be examined, inspected and tested in a manner and at a frequency which is adequate to confirm that the structural integrity, performance and reliability claims made in the safety case continue to be met throughout the operational life of the station. The performance criteria for nuclear safety related structures must be determined on the basis of the duty required of each structure at each of the stages of the lifetime of the facility. Consequently, detailed procedures exist for the routine inspection of these structures in normal service.

In the event of a fire on a nuclear power plant the standard inspection procedures may no longer be applicable and the Licensee would have to demonstrate that any nuclear safety related structures in the area of the fire are still fit for purpose and capable of meeting their nuclear safety role.

Following a fire in an area that could potentially affect the condition or performance of a structure, a specific procedure would be written to cover the examination of that structure, the method and criteria for its assessment and acceptance criteria for its continued use. The specific post-fire inspection procedure would be based on the normal in-service inspection procedure amended to take account of the extent of the damaged area. All areas where structural failure could cause damage to, or failure of, nuclear safety related equipment would be addressed. The resulting inspection report would include lists of the defects found and give recommendations on any remedial works required.

This paper considers the factors that would be applicable to specific post-fire inspection procedures for the inspection and assessment of nuclear safety related structures.

Post-Fire Damage Assessment Procedures for Nuclear Power Plant Structures

Introduction

As fire is an identified risk on Nuclear Power Plants (NPPs), it is considered at the design stage and preventative measures, segregation and suppression systems are built into the plant design in order to limit the effects of fire should it occur. However, although this greatly reduces the likelihood and effects of potential fires it does not totally remove the risk of one taking place.

This paper is based on circumstances as they exist in the UK but the general principles are equally applicable to other locations. Should a fire occur, the post-fire assessment of structures on UK NPPs would be treated as a special investigation not covered by normal standing procedures. This document only provides a brief over view of the subject. For more detailed information on the effects of fire on the structural materials involved the references given at the end of the paper should be consulted [2-10].

General Background

Each site in the UK is covered by a Nuclear Site Licence which contains 36 standard conditions which must be met by the operator and is issued under the provisions of the Nuclear Installations Act 1965 by HM Nuclear Installations Inspectorate. The rules are not prescriptive and the licensee retains absolute responsibility for nuclear safety under UK law. [1]

The regulatory requirements have major implications for the in-service inspection regime, principally through site licence condition 28, which covers the examination, inspection, maintenance and testing of all plant and structures that may affect nuclear safety. It is essential that all nuclear safety related structures must be maintained throughout their operational life in such a way that they are always fit for purpose and capable of meeting their nuclear safety role as required by a detailed safety case, which is identified in the licence conditions. In order to do this they must be examined, inspected and tested in a manner and at a frequency which is adequate to confirm that the structural integrity, performance and reliability claims made in the safety case continue to be met throughout the operational life of the station [2]. The performance criteria for nuclear safety related structures must be determined on the basis of the duty required of each structure at each of the stages of the lifetime of the facility.

In normal operation, a list of nuclear safety related structures and procedures for the inspection of each of them are prepared from examination of the Station Safety Report. The inspections are visual in the first instance and checklists are prepared for each building or area of structure from detail drawings and preliminary inspections that give guidance to the inspection team.

Under the terms of Site Licence Condition 28, after a fire, the Licensee would have to demonstrate that any nuclear safety related structures in the area of the fire are still fit for purpose and capable of meeting their nuclear safety role as required by the safety case. In the event of a fire in an area that could potentially affect the condition or performance of a structure, a specific procedure would be written to cover the examination of that structure, the method and criteria for its assessment and acceptance criteria for its continued use. As in the assessment of damaged areas of conventional structures, personnel safety would be of high importance and require a risk assessment to be carried out but on NPPs there is the additional requirement to ensure radiological safety. This will require survey and monitoring of the area and a written system of work to keep any potential radiological dose to personnel to a level that is as low as reasonably practicable.

The specific post-fire inspection procedure would be based on the normal in-service inspection procedure amended to take account of the extent of the damaged area. All areas where structural failure could cause damage to, or failure of, nuclear safety related equipment would be addressed. The resulting inspection report would include lists of the defects found and give recommendations on any remedial works required. IAEA Safety Report No.8 (Preparation of fire hazard analyses for NPPs) [11]covers the need for a fire preplan. It may be argued that there is also a need for a fire postplan prepared in advance to cover the procedures and methods to be used to assess the immediate safety of nuclear safety related structures (in the order of their importance to nuclear safety) following a fire.

On British Energy NPPs, defects are defined in three nuclear safety categories; Category 1: Affecting safety - repair required immediately; Category 2: Not affecting safety - repair required as soon as possible to prevent further deterioration; and Category 3: Not affecting safety - repair carried out under normal station maintenance programme. The categorisation is based initially on an assessment made by the inspection engineer, taking into account the current safety case and overall structural integrity. Identified defects would be included (with normal inspection defects) in a monitoring database which allows easy progress tracking and identification of faults.

In addition to the defect classification given above, a damage classification of the effects of the fire on the structural members should be carried out to allow a structural assessment to be carried out. Tabulated damage classifications are useful in this regard (Table 3 [9]).

When assessing a fire damaged structure the points given below should be taken into consideration.

Fire Damage Assessment

Where possible detailed information concerning the use and occupancy of a structure should be obtained prior to a post-fire investigation to enable an estimate of the fire loading to be determined. Station Logs, Fire brigade reports and eyewitness accounts are useful for determining the duration and course of the fire. Before the structural investigation is carried out, as-built construction drawings should be obtained showing all the main members of the structure and the form of construction. Care must be exercised to ensure that the effects of all previous plant modifications and repairs have been included in the information collated [3]. Previous in-service inspection reports can provide useful information on the pre-fire condition of the structure and normal in-service inspection procedures identify critical areas of the structure.

A variety of different methods, ranging from visual survey through to non-destructive examination and comprehensive instrumentation, is available to the engineer to allow condition monitoring and lifetime management of structures to be implemented. Structural investigations should, where possible, utilise non-invasive methods such as visual surveys or non-destructive testing although there are limitations to the information that may be obtained from such sources. Where it exists, instrumentation may provide useful information on the performance of materials and structures. Invasive investigations involving the removal of a sample or specimen for testing should only be used where simpler methods cannot provide the required information. Careful attention must be paid to the limitation of damage caused during sample collection and reinstatement of the structure must be given detailed consideration and executed correctly.

The method of investigation that is chosen in any particular post-fire situation for the assessment of fire damage to concrete structures will obviously depend on the accuracy of the results required. Each method has its advantages and disadvantages but every method, if properly used with sufficient care, may give an assessment of the fire damage to the structure. No one method is entirely free from error and in most cases a combination of tests will be used depending on the importance and cost of the reinstatement of the structure, the time allowed for repair and whether demolition and replacement is a feasible option.

The examination of fire debris is a very important stage in any post-fire investigation as it allows an overall picture of fire severity to be gained by the investigator. This is especially useful if combined with visual damage classification and a sounding survey of the structure. The main drawback with debris surveys is that the final position of thermal indicators may bear little relationship to their position at the time of the fire if any debris clearance has taken place before examination.

Pre-fire defects must be isolated from fire damage and noted specifically as such along with other influences such as blast or explosion damage (see [3, 9]).

Photographs are a most important part of any post-fire survey as they provide a visual record and may yield useful information at later stages during investigation or reinstatement should problems be encountered.

Concrete Structures

Colour changes in concrete and aggregates may be used as a valuable guide in fire damage assessment in cases where significant changes occur due to thermal exposure. In some instances, however, a lack of colour change may not be taken as inferring that concrete is unaffected by high fire temperatures and care should be exercised especially in cases where ingneous aggregates have been used.

Ultrasonic pulse velocity (UPV) measurements may be employed in damage assessment if a sound area of concrete is available for comparison and the physical configuration of the member and its reinforcement and the surface condition of the concrete are suitable. In complicated cases, laboratory analysis of the results may be necessary to interpret the readings. Any UPV survey must be preceded by a cover meter survey to identify the position, orientation and size of steel reinforcement.

Cores may be taken from concrete structures to allow the depth of colour change to be determined or for material testing purposes. However, as the damage will be worst at the fire exposed surface, which will be likely to be in the area of the core nearest the plattens of the testing machine, care must be taken when interpreting the test results. Coring should be carried out in non-critical areas of structures where stress levels are low.

Thermoluminescence (TL) measurement provides an excellent indication of the thermal exposure that concrete has been subjected to and from this the residual compressive strength may be judged. As with UPV testing, however, it must be realised that concrete strengths may occupy a range of values for any thermal exposure and only an approximate figure may be attributed to the residual strength. TL testing is particularly valuable in critical areas of a structure where other methods may not be applied. The TL test is the only test that can tell definitely if concrete has been heated significantly and in many cases its most useful application is in indicating areas that have not been subjected to damaging thermal exposures (especially in cases where a large amount of smoke damage is present). Post-fire exposure to radiation may affect the TL signal.

In prestressed concrete structures, such as containments and pressure vessels, the residual level of prestress will be of great importance. Installed instrumentation may be used if it has survived the fire and in un-bonded prestressing systems load checks may be carried out and tendons withdrawn for inspection and testing. In this type of system it is possible to replace and restress tendons that may have been affected by high temperatures in order to restore the level of prestress in the structure. The situation with regard to bonded prestressing systems can be more difficult to assess as withdrawal, replacement and restressing cannot be carried out.

Brickwork and Blockwork

By virtue of their method of manufacture, clay bricks perform well under fire conditions although the mortar used in wall construction and concrete blocks will degrade in a manner similar to that of unreinforced concrete. Brick and blockwork panels may also be damaged by restrained thermal forces and by global structural movement of steel or concrete frames and by surface damage due to shock cooling during fire fighting operations.

Steelwork

In general, a structural member remaining in place, with negligible or minor distortions to the web, flanges or connections should be considered satisfactory for further service. The exception will be for the relatively small number of structures in cold-worked or tempered steel where there may be permanent loss of strength. The change in strength may be assessed using estimates of the maximum temperatures attained or on-site tests (such as hardness tests); if necessary, the steel should be replaced. Microscopy can be used to determine changes in microstructure. Since this is a specialised field, the services of a metallurgist are essential [9]. Bolted connections may require detailed consideration if heated above 360°C.

Supplementary Testing

Supplementary testing such as load testing or detailed materials testing is normally only required in special cases where the damage assessment is highly critical or where special problems exist. This may include tensile and hardness testing of steel, measurement of the modulus of elasticity of the material, durability testing or load testing of the entire structure or a portion of it. If a fire has occurred inside a containment structure, a structural overpressure test may be required to confirm the structural behaviour and leaktightness of the containment.

Once the residual characteristics of the structure have been determined, a design check based on the material properties must be carried out to examine the ability of the structure to continue to fulfil its design role and to identify areas in need of strengthening.

Further Information

Further detailed information on the effects of fire on structures and structural surveys may be obtained by consulting the references given at the end of this document [3-10].

Table 1 gives details of a typical outline procedure for the post-fire inspection of NPP structures and Table 2 gives details of temperature indicators which may be used to assist the investigation.

Recommendation

Although detailed inspection procedures should be written for each structure after a fire event, it is recommended that an outline higher level generic fire postplan is written which contains a ranking of nuclear safety related structures (in order of their importance to nuclear safety) and outlining general procedures for post-fire inspection in order to reduce the response time.

References:

1. "The Nuclear Installations Act 1965", HMSO, UK, 1965 (*et seq.*)

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Table 1 Post-Fire Investigation (modified for NPPs) [3]

Stage	**Activity**		
1	**Preparation** Obtain (i) Original design calculations and as-built drawings (ii) Plant modification details (iii) Nuclear Safety Case (iv) Normal in-service inspection procedures and previous survey reports (v) Station log (vi) Fire Brigade reports (vii) Eye witness reports (viii) Any available monitoring instrumentation results Write (i) Specific post-fire inspection procedure (ii) Specific written system of work/radiological documentation & requirements		
2	**Visual Examination** (i) Identify any pre-fire damage (eg settlement) (ii) Locate seat of fire (iii) Chart progress of fire (iv) Examine debris for maximum temperature (v) Locate special damage (eg explosion damage) (vi) Locate areas of above average fire damage (vii) Classify members (viii) Take photographs		
	CONCRETE	Accuracy❶	Time❷
3	**Simple In-situ Testing** (i) Sounding (hammer survey) (ii) Colour Changes (if applicable)	1 2-3	A A
4	**More Complex In-situ Testing** (i) UPV Measurement (ii) Cores for colour examination (iii) Deflection measurement	2-4 2-3 4	B-C B-C B-C

Table 1 (continued)

5	**Laboratory Testing**		
	(i) Cores for strength		
	(ii) Themoluminescence	3	C
	(a) For thermal exposure		
	(b) For residual strength	4	C
	(iii) Materials testing (other than 5(i))	2-4	C
	(eg Reinforcement & prestressing tendons)	3-4	C-C+
	STEELWORK		
6	**In-situ Testing**		
	(i) Distortion & Deflection		
	(ii) Hardness testing	1-3	A-B
	(iii) Metallographic investigation	4	A-B
		4	C-C+
	GENERAL		
7	**Supplementary Testing**		
	(i) In-situ load testing (incl. SOPT & Tendon Load)	-	C+
	(ii) Design check	-	C+
8	**Final Report** Including photographic record, repair recommendations and update to safety case		

Post-Fire Investigation Guide (See Table 1)

❶ Accuracy

1 General Guide
2 Limited Accuracy
3 Moderately Accurate
4 Accurate

❷ Time

A Relatively quick in-situ examination
B Time consuming in-situ examination
C Requires lab analysis and/or time to recover specimens/analysis

Table 2 Post-fire Thermal Indicators [3, 9]

Approximate Temperature, ºC	Indicator & Condition	Typical Example	Method of Observation	Class
100+	Paint deteriorates	Coatings	Visual	I, II
120	Polystyrene items collapse, polythene items shrivel, PVC degrades	Plastic items, cable insulation	Visual	I
120-140	Polystyrene softens, polythene softens and melts	Plastic items	Visual	I
130-200	Polymethyl methacrylate softens	Handles, covers, glazing	Visual	I
140	Polyurethane foam charred black	Thermal insulation	Visual	I
150	Paint destroyed	Coatings	Visual	I
150	PVC fumes	Cable insulation	Visual	I
150-180	Polystyrene melts and flows	Plastic items	Visual	I
170	Phenolic resin changes from yellow to brown	Wall linings, roof sheets	Visual	I
180	60Sn-40Pb solder melts	Solder joints - electrical equipment	Visual	I
200	PVC browns	Cable insulation	Visual	I
205	Charring & clay like appearance of acrylic resin	Thermal insulation	Visual	I
250	Charring of wood begins	Doors, floors, furniture fittings	Visual	I
250	Polymethyl methacrylate bubbles	Handles, covers, glazing	Visual	I
275	Lead base babbitt melts	Sliding bearings in pumps & compressors	Visual	I
280	Copper instrument tubing begins to soften, recrystallise	Instrument tubing	Hardness test	II
300-350	Lead, sharp edges rounded or drops formed	Plumbing, fixtures, shielding	Visual	I
390-400	Zinc die casting alloy melts	Plumbing fixtures, small components	Visual	I
400-500	PVC chars	Cable insulation	Visual	I
420	Zinc coating melts	Galvanised steel	Visual	I
450-870	Austenitic stainless steel sensitised	Piping	Metallographic	II
480	Asbestos powders/flakes	Column packing	Visual	I
540	High temperature scaling begins on carbon steels	Carbon steel exposed to air	Visual	I
595	Bolts tempered to lower than normal hardness	ASTM A193 B7 & B16 bolting	Hardness test	II

Indicator Classes - I Unaffected by time (continued)

II Events of a more complex nature involving functions of time, temperature & cooling rate

Table 2 (continued) Post-fire Thermal Indicators [3, 9]

Approximate Temperature, ºC	Indicator & Condition	Typical Example	Method of Observation	Class
600-650	Aluminium & alloys melt	Small machine parts, brackets, electrical conduits	Visual	I
700-750	Sheet glass softened or adherent	Glazing	Visual	I
750	Moulded glass rounded	Corrugated window glass	Visual	I
760	Gross deformation of low carbon steels	Structural steel members	Visual	I
760	Inorganic zinc paint darkens, spalls off	Structural coatings	Visual	I
800	Sheet glass rounded	Glazing	Visual	I
820	Borosilicate glass softens, melts	Instrument gauges, sight glasses	Visual	I
845	Bolting hardened well above normal range	Steelwork connections	Hardness test	II
850	Sheet glass flowing easily	Glazing	Visual	I
900-1000	Leaded red brass/ brass melts	Plumbing fixtures	Visual	I
905	Zinc coating boils off	Galvanised steel	Visual	I
950	Ni/Au bronze metal melts	Thermocouple wave rings	Visual	I
980	Foamglass insulation melts to a black-grey slag	Thermal insulation	Visual	I
1000-1100	Copper melts	Wiring	Visual	I
1100-1200	Cast iron melts	Castings	Visual	I
1400	Low carbon steel melts	Structural steelwork	Visual	I

Indicator Classes - I Unaffected by time

II Events of a more complex nature involving functions of time, temperature & cooling rate

Table 3 Classes of Post-fire damage, Characterisation and Description [9]

Class	Characterisation	Description
1	Cosmetic damage, surface	Characterised by soot deposits and discolouration. In most cases soot & colour can be washed off. Uneven distribution of soot deposits may occur. Permanent discolouration on high-quality surfaces may lead to rejection. Odour is included in the class; it may be difficult to remove but chemicals are available for elimination.
2	Technical damage, surface	Characterised by damage to surface treatments and coatings. Small extent only of concrete spalling or corrosion on uncovered metals. Painted surfaces can be repaired. Plastic coated surfaces need replacement or covering. Minor spalling may remain or can be re-plastered.
3	Structural damage, surface	Characterised by some concrete cracking and spalling, lightly charred wood surfaces, some deformation of metal surfaces or moderate corrosion damage. This type of damage includes Class 2 damage and an be repaired similarly.
4	Structural damage, cross-section (interior)	Characterised by major concrete cracking and spalling, deformed flanges and webs of steel beams, partly charred cross sections of timber constructions, and degraded plastics. Damage can in many cases be repaired on the existing structure. Within the class are also deformations of structures so large that the loadbearing capacity is reduced, or dimensional alterations prevent proper fitting into building. This applies in particular to metal/steel constructions.
5	Structural damage to members and components	Characterised by severely damaged structural members and components, impaired materials and large deformations. Concrete constructions are characterised by extensive spalling, exposed reinforcement and impaired compression zone. In steel structures extensive permanent deformations have arisen due to diminished loadbearing capacity caused by high temperature conditions. Timber structures may have almost fully charred cross sections. Changes in materials may occur after fire, so they may display unfavourable properties. Class 5 damage will usually lead to rejection.